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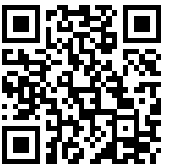

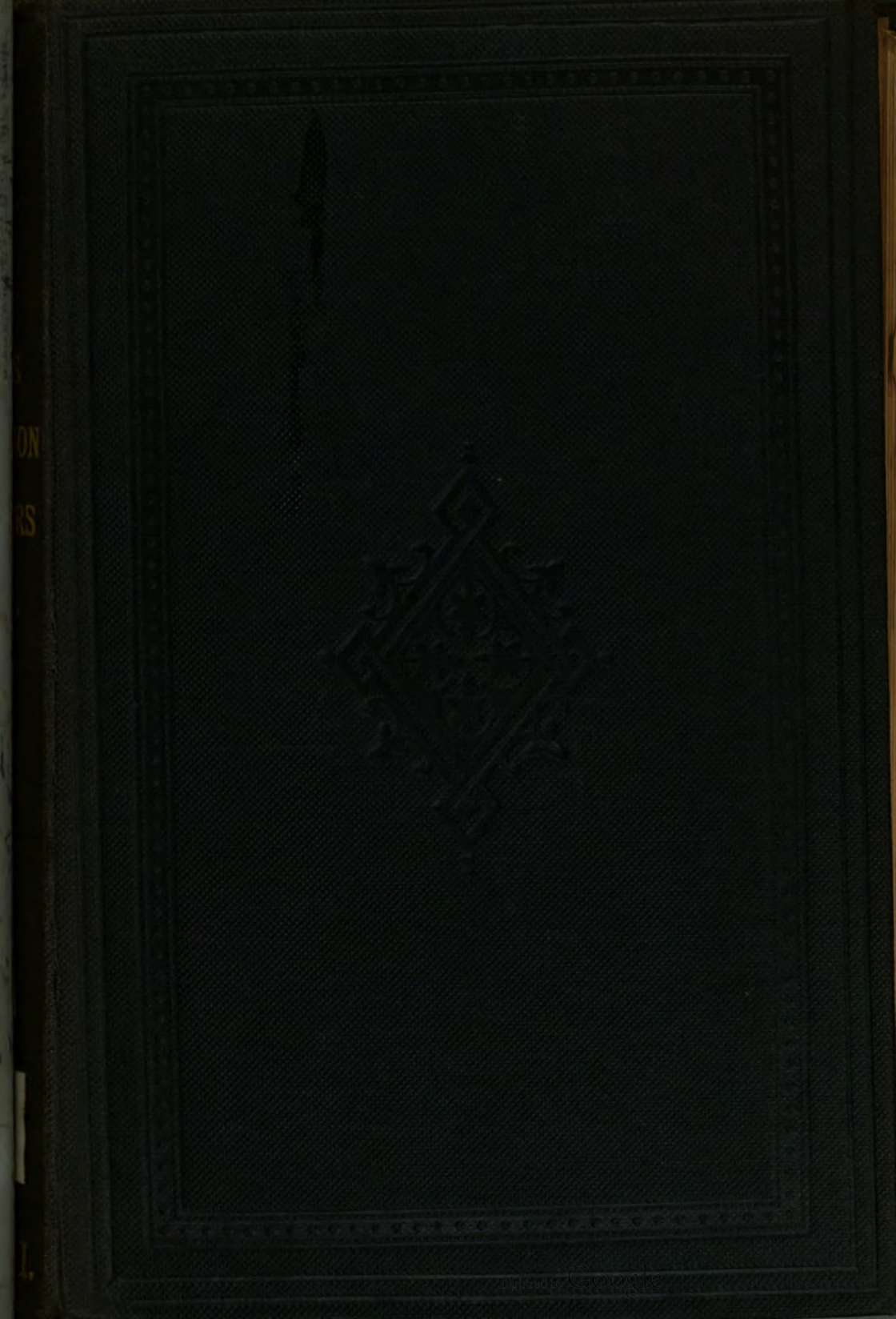

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MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS.

VOL. 221.

EDITED BY

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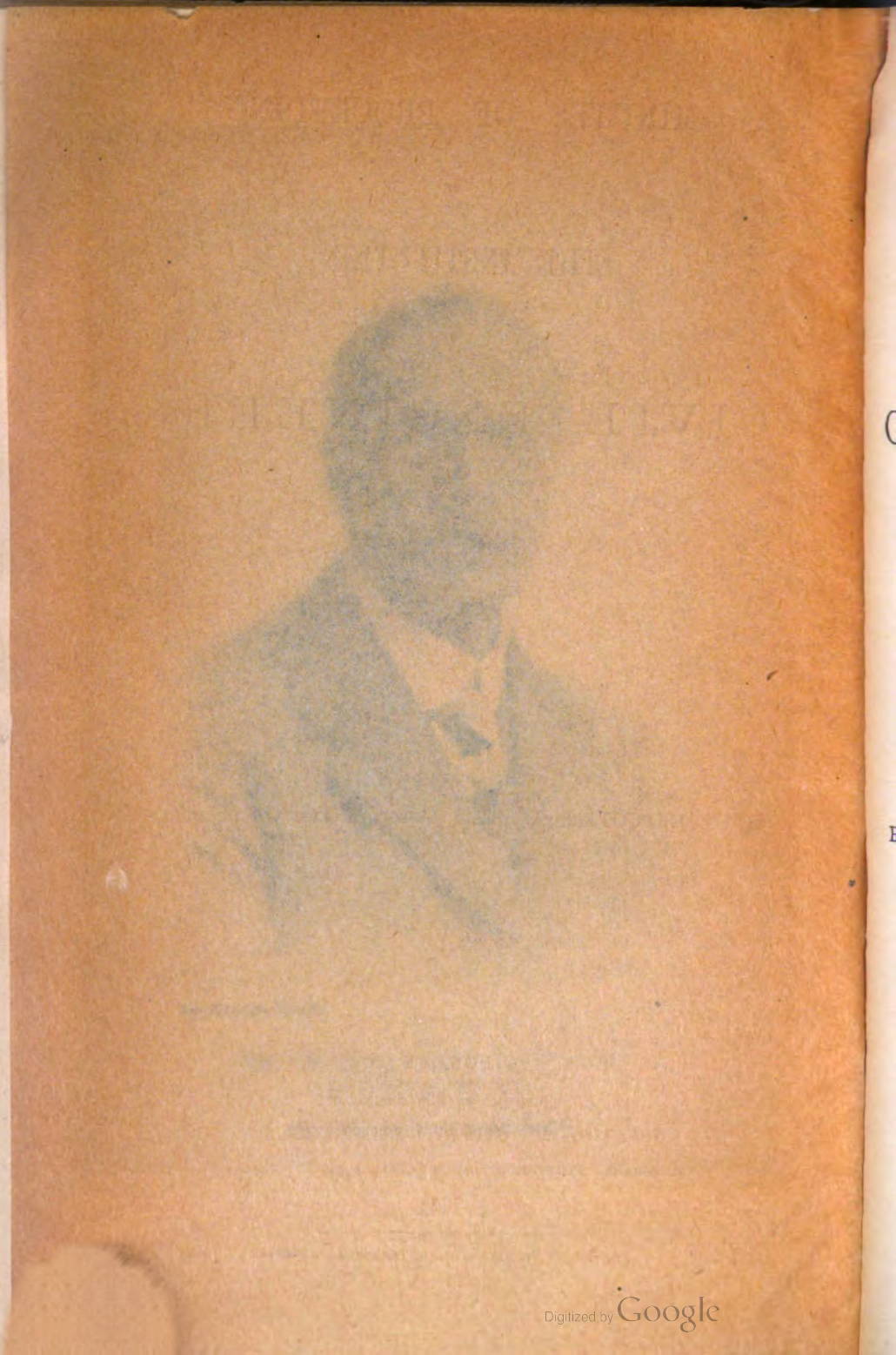

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THE

INSTITUTION

OF

CIVIL ENGINEERS.

SESSION 1925-1926.—PART I.

MINUTES OF PROCEEDINGS.

3 November, 1925.

BASIL MOTT, C.B., the retiring President, in the Chair.

Mr. MOTT said it was with the deepest regret that he had to record the deaths of two distinguished men associated with The Institution, namely, Viscount Milner, Honorary Member, and Captain Sankey, a Member of Council.

Viscount Milner's death was a loss to the whole Empire. His name was known throughout the world, and would be remembered for many generations to come. The Council had passed the following resolution :—

> " That the Council record the deep regret with which they have learned of the death of Viscount Milner, K.G., G.C.B., G.C.M.G., P.C., M.A., D.C.L., a distinguished Honorary Member of The Institution."

By the death of Captain Sankey The Institution had lost one of its most able men, and one whose place it would be very difficult to fill. His life was largely devoted to an endeavour to promote the interests of The Institution and of all other Institutions to which he belonged. Among other things he was Chairman of the Heat-Engine Committee which was brought into being largely by his efforts and had given results of great value ; and one of his last important services to The Institution was to deliver in May of that year, an admirable James Forrest lecture. In addition to his great scientific attainments, he was a man of the highest character ; a soldier from his early days, he retained throughout his life the highest ideals of a soldier. He was the very soul of honour, courage, and helpfulness. The Council had passed the following resolution :—

> " That the Council record their deep regret at the death of Captain Matthew Henry Phineas Riall Sankey, C.B., C.B.E., R.E. (ret.), whose long association with The

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Institution and continuous and active interest in its welfare was marked by his election as a Vice-President at the last Annual General Meeting.

"The Council desire that an expression of their regret and condolence with Captain Sankey's family be conveyed to Mrs. Sankey."

He was sure the members would wish to associate themselves with the Council in those resolutions.

It was now his pleasant duty to introduce the new President, Sir William Ellis, although no introduction was really required, because Sir William's work was so well known. He was sure that Sir William Ellis would add lustre to the Presidential Chair.

Sir WILLIAM ELLIS, G.B.E., D.Eng., President, having taken the Chair,

Dr. W. C. UNWIN said that it fell to him, as the senior Past-President present, to perform the duty of proposing the thanks of the members to Mr. Basil Mott for the way in which he had carried on the business of The Institution during his year of office. He was sure that all those who were present would know with what courtesy and dignity Mr. Mott had presided over the meetings of The Institution. Those who were on the Council knew how assiduous he had been in attending to the not small business which fell upon a President during his year of office. He had maintained the traditions of The Institution and had enabled advance to be made without any revolutionary movement; and no detail of business had been neglected by him, although he had very serious responsibilities on his shoulders. He had very great pleasure in moving:—

"That the members present at this meeting desire on behalf of themselves and others to record their high appreciation of the services rendered to The Institution by Mr. Basil Mott during his term of office as President."

Sir JOHN A. F. ASPINALL, Past-President, in seconding the vote of thanks, said The Institution had been extremely fortunate in having Mr. Basil Mott as President during the past year. There was little doubt that his unfailing courtesy, his ceaseless application to work, and his willingness to give up a very large amount of professional time to the interests of The Institution, had earned, and more than earned, the thanks of the members; and he asked those present to recognize that in the heartiest manner possible.

The motion was carried by acclamation.

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Mr. BASIL MOTT thanked the meeting very much for the way in which the resolution had been received. It was particularly gratifying to him that it should have been moved by Dr. Unwin and Sir John Aspinall. Dr. Unwin occupied a unique position in the hearts of all; he was the most revered member of The Institution; He was bound to say that he left the Presidential Chair with regret. It had its responsibilities, but he had received great kindness and much consideration from the Members of Council and from very many members of The Institution of which he was so proud. He would not say more than to thank them, and also to thank the officers, the Secretary, Dr. Jeffcott, and Mr. Spear, for the great help they had given him during his year of office. He thought no President could possibly be of any value to The Institution without the great assistance he derived from the loyal support of the Council and the staff.

The PRESIDENT then delivered the following Presidential Address.

GENTLEMEN,

Election to the Office of President of such an Institution as ours arouses feelings of a very mixed character in the honoured individual. After many years of membership of the Council, he is conscious of the magnitude of the responsibility he has undertaken, and he wonders whether it is possible for him not only to fill the Office to the satisfaction of his fellow members, but also to maintain the high ideals which are the aim of our Institution.

I thank you for electing me as your President, and I shall do all I can to justify your confidence.

I take office as President with credentials very different from those of many of my predecessors. I cannot lay claim to personal association with the production of any great reservoir or bridge or railway work. The forty-seven years of my engineering life have been largely spent in steelworks, dealing with the varied and difficult problems connected with the great development of mechanical engineering during the last 40 years; and, indirectly, I have been associated with mining and marine engineering.

Former Presidents, I feel, have been generous in devoting their addresses largely to the subject of their life's work, and I shall, as far as possible, follow their example. I propose speaking to you shortly on four points, namely: steelworks engineering, colliery engineering, training for engineers, and standardization and specifications.

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Steelworks Engineering.

After a varied apprenticeship in Leeds, coupled with considerable experience in this country and abroad in erecting plant, I was entrusted with the installation of one of the very early plants for hydraulic forging, an industry which appeared to me likely to develop, as it introduced into steelworks a class of engineering of an entirely novel description; and I was offered, and accepted, an engagement in one of the large steelworks taking up this type of work.

In those early days opportunities for technical training for engineers were very limited, and in the various steelworks engineering was in the hands of good practical men who could only with difficulty move with the times, owing to their lack of knowledge of the technical side of engineering. Low-pressure Lancashire boilers supplied steam to distant mill and machine-shop engines, with inevitable waste in condensation. The Bessemer process of steel-making was already in use, and the Siemens process also to a smaller extent; and these had brought with them a considerable demand for hydraulic power, generally at about 400 to 800 lbs. per square inch. So far as I know, electricity was not then applied in any of the large steelworks. All large machine tools were belt driven, without knowledge, on the part of either maker or user, of the horse-power required, beyond the fact that a belt of a certain width and tightness would drive a machine carrying certain cuts. High-speed tool steel was unknown, except Mushet steel, which was used more for its power of holding a cutting edge against unexpectedly hard material than as high-speed tool steel is now used, namely, for removing a heavier weight of material in a given time.

The new problems caused by the more scientific development of engineering presented difficulties to the works engineer of those days. He did not understand the principles of electricity, and failed to realize that, once understood, they would help him with many of his difficulties. He rather feared the introduction of higher steam-pressure, not realizing that safety depended on suitable design. He had to design his own structural work, instead of being able to send general dimensions to structural engineers with efficient staffs who specialize in this class of work, as is the case to-day.

There was, therefore, a great opportunity for young engineers, who could combine a measure of technical training with practical, to take positions in steelworks, and share in the work of changing the character of engineering by introducing more modern machinery.

University colleges, with special courses for engineers, began to appear in the industrial areas of the country, and took a

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great share in supplying the technical training so badly required. Fortunately, in those early days economical production was not as important as in our own competitive times. Our country had a good start both in railway development and in shipbuilding, and there was comparatively little foreign competition—a very different state of affairs compared with the present time.

The introduction of much higher steam-pressures in steelworks, with water-tube boilers, and the advent of electricity, have been the two factors of the greatest help to steelworks engineers. The comparison, with modern practice, of the old method of driving cranes or machinery from boilers of 50 lbs. per square inch pressure, with long steam-mains to engines in the different departments, each in turn driving long lines of shafting from which the machines were belt-driven, is indeed interesting; and I hesitate to attempt to set out the economic saving resulting therefrom. I have to thank my colleague, Mr. Maurice E. Unwin, M. Inst. C.E. (a kinsman of our honoured member Dr. Unwin), for preparing for me some comparative figures. I have compared what was common practice 40 years ago with reasonably efficient modern practice, and the result will no doubt appear grotesque to many of our members who have been brought up on the more enlightened methods. The comparison is, I believe, a fair one, and does not at all exaggerate the economy that modern engineering methods have brought into steelworks. *Fig. 1* (p. 6) illustrates graphically the changed conditions, due to the adoption of electric driving, to which I have referred. In the three examples shown, the total height of the column in each case represents the quantity of coal consumed to produce equal quantities of useful work, represented by the shaded area at the top of each column. I have assumed a calorific value of 13,000 B.Th.U. per pound of coal, which is a fair figure for Yorkshire coal, but is somewhat higher than the average quality used in power-stations throughout the country.

Examples A and B show actual results obtained in the case of a large machine shop 30 years ago, and at the present time.

In A, with low-pressure boilers, long steam-mains and line shafts, and inefficient prime movers, the energy available for useful work was only 1·5 per cent. of the total thermal energy in the coal consumed. In B, with a modern central power-station of moderate size, electric transmission, and motor drives for individual machines, the useful energy obtained has risen to 10·43 per cent. In example C it is assumed that the power is derived from one of the latest and most efficient super power-stations in this country, giving, according to the Commissioners' Returns, an overall thermal efficiency of 20·33

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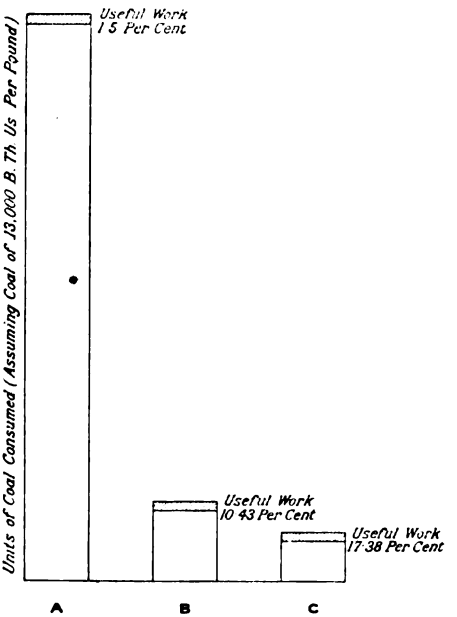
per cent. Allowing for cable and motor losses, as before, we find that 17·38 per cent. of the heat energy of the coal consumed could be delivered to the machine tools.

The saving in fuel which has been effected requires no emphasis, but, as engineers, we cannot yet feel satisfied, for, even under the most modern conditions, we are only able to transform less than one-fifth of the thermal energy of coal into useful work of this description.

Fig. 1.



IMPROVED EFFICIENCY DUE TO ADOPTION OF ELECTRIC DRIVING IN A LARGE MACHINE-SHOP. THE COAL CONSUMED INCLUDES ALL LOSSES IN GENERATION AND TRANSMISSION UP TO THE DRIVING-SHAFTS OF THE MACHINES.

The great strides and progress of marine engineering, requiring larger horse-power, and the introduction of turbine driving, have brought many new problems to the steelworks engineer. Large-diameter shafts have required larger hydraulic presses for their manufacture, and the Parsons turbine, and subsequently gear-rims therefor, caused the introduction of hollow rolling. Up to the time of the S.S. " Lusitania " turbine-drums were nearly all welded, and I can assure you that this was very difficult work, causing not a little anxiety; but it was my good fortune to be able to apply a

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process of punching and hollow rolling, until then used only for tube-making, to the much larger problem of producing weldless hollow rolled drums in some cases more than 30 tons in weight. A mill specially designed for this purpose was installed; it has revolutionized this important industry, and the results have been entirely satisfactory. What has always been wanted for this purpose has been a satisfactory ingot, cast hollow. I am interested at present, with my engineering and metallurgical colleagues, in experiments in this direction by means of centrifugal casting. It is certainly a most fascinating process, with many difficult problems not yet entirely solved; and I mention it as an instance of the importance of the closest possible co-operation between the metallurgist and the engineer in the various steelworks.

The very powerful engines now required for warships, ocean liners, and electric power-stations, have brought many interesting problems to the engineering staff in a steelworks. Also the introduction of much higher boiler-pressures, which were alluded to in one of our Papers last session, brings with it important questions requiring careful attention on the part of engineers engaged in this class of work. Under normal circumstances the usual tests of steel in the cold state may be regarded as representing the physical condition of the steel when at work, but at the temperature corresponding to 500 lbs. per square inch pressure, and superheat, a figure of 750° F. and even more may be reached, and at this temperature the physical properties of steel begin to change. I am carrying out some research on this point with my metallurgical colleague, Dr. W. H. Hatfield, and we are making a series of tests at different temperatures, the results of which will be useful to engineers who are dealing with these very high steam-pressures. If still higher pressures, with their corresponding temperatures, are introduced, it may even be necessary to use alloy steels, the physical properties of which are only affected at much higher temperatures.

On the subject of the use of steels of special qualities it is very necessary for engineers to realize how delicate some of the high-stress alloy steels are, and how carefully heat-treatment must be applied to them. They believe, quite rightly, that the nickel, chrome, or other alloy steels will give them material of a quality better suited to their requirements, but they do not always realize that such steels, without exact heat-treatment varied accurately according to the analysis, will give unreliable results, inferior even to those of ordinary carbon steel, and that their use will introduce dangers of cracking and uncertain physical results unknown in ordinary mild steel. Consultation on these questions is, therefore, most desirable.

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A considerable amount of research work is being carried on by steelworks metallurgists on the production of steels able to resist corrosion in sea-water, and I have sent forward valuable data on this subject to Mr. Maurice F.-G. Wilson, M. Inst. C.E., the Chairman of our Special Committee. This is a question, I think, of great importance to those of our members who are concerned with sea works. One further point before I leave this subject. The handling without accident of large weights of molten steel, 70 to 80 tons, excluding the weight of the ladle, and the carrying about for forging purposes of 80 to 100-ton ingots, involving, with porter bar, a weight of nearly 200 tons, are matters of considerable responsibility for the steelworks engineer, and it speaks well for his training, and for the careful design and maintenance of the machinery in his charge, that we have to go back for many years to find any serious accident involving loss of life.

I would urge on the heads of steel firms the desirability of encouraging the members of their staffs to become members of one or other of the Scientific Institutions such as our own, so that they may have the opportunity of reading the Papers and discussions, and thereby receive much inspiration and practical help in dealing with new developments.

Colliery Engineering.

I use this heading as I want to speak of mining engineering in its wider aspect. I find it is more than 20 years since a President alluded to this subject in his Address; and, realizing that we have many distinguished members in this branch of the profession, I should like to speak of the progress of recent years, with which I have to some extent been associated. We must all hold these members in respect, for no engineers are more definitely entrusted with the lives of their subordinates. How very varied is the knowledge required by these engineers; for it includes surveying, pumping, ventilation, electricity, maintenance of permanent way and of thousands of main-line wagons, house-building, chemistry in its many applications to by-products, mechanical engineering—also involving winding and underground haulage—and design and maintenance of screening plant, in addition to the variety of circumstances met with in mining operations. I have, in a Paper I gave recently to a mining society, pointed out how difficult it is for the mining engineer of a large colliery to be conversant with all the modern developments in mechanical engineering, for, in addition to his purely technical subjects mentioned above, he has labour

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matters and commercial questions also in his hands. Immense progress has been made in recent years in colliery engineering, but more remains still to be done, and with such a diversity of matters to be dealt with by the mining engineer the task of maintaining colliery engineering at the same level of precision and efficiency which other branches such as marine and electrical engineering have attained, is indeed a severe one. Having pointed out somewhat fully the very varied and responsible duties of the mining engineer, I want to urge the desirability of introducing well-trained mechanical engineers on to his staff. They will not touch purely mining matters, but they can be of immense help to their chief in preparing and working out in detail various mechanical improvements, for the study of which their training should especially fit them. In any large colliery plant the presence of a first-class mechanical engineer taking charge of the machinery, and thereby relieving the mining engineer in this respect, must result in economical maintenance of plant and will fully justify the expense involved.

Electricity has, as yet, been introduced only to a limited extent, and the question how far in-bye it may be taken is still uncertain, and must always vary according to the individual colliery and its liability to outbursts of gas. No doubt electrical engineers may aid its use by further reducing the liability to sparking. The immersion of the switches in oil in closed boxes is no doubt a considerable safeguard, and the well-thought-out design of main cables now in use appears to me to afford great security against risk of short-circuiting in cases of falls of roof.

Electric haulage in the workings has largely replaced the very uneconomical and undesirable practice of having engines on the surface with haulage ropes in boxes down the shafts. Electric winding is being introduced, and, with the efficient means of control and deceleration now being adopted, is making considerable progress. This method has many points in its favour, and but with my limited knowledge I do not feel competent to discuss it. Efficient winding machinery is, of course, a matter of great importance, and the problem is an interesting one. Taking a typical instance, a load of 30 tons is started from and brought to rest in 60 seconds, travelling a distance of 2,250 feet and reaching a maximum speed of more than 50 miles per hour. The acceleration and deceleration are of an unusual nature, and the resultant diagram is one of interest. Although to our mining-engineer members this is all well known, I give a diagram (*Fig.* 2, p. 10) as I feel it may interest members in other branches of engineering.

This diagram is naturally prepared from a series of indicator-

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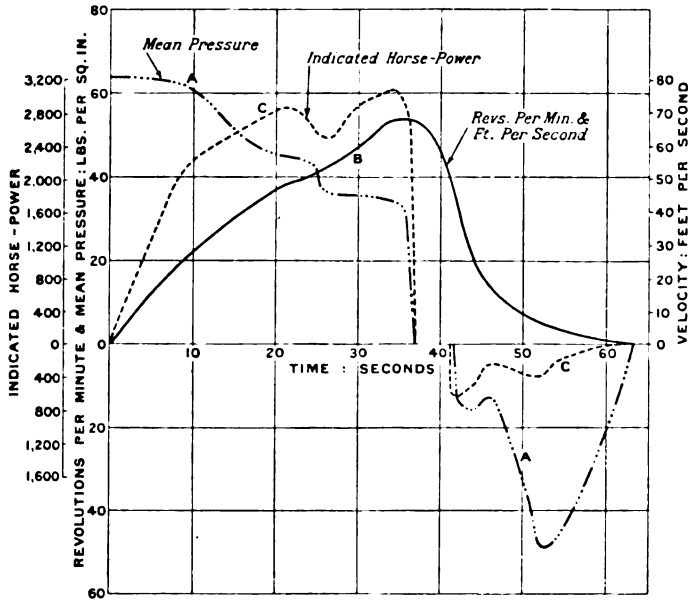
diagrams taken from a winding-engine throughout the run of the cage from pit-bottom to the surface. The starting of a 30-ton load from rest with rapid acceleration naturally produces a series of diagrams similar to those of a locomotive starting a train from rest. These diagrams improve very much after line A has crossed lines C and B, but I think a very interesting point in the diagram is where the steam is shut off entirely at about two-thirds of the run. Although the engine is not reversed in its running at this point, the steam is immediately put against the piston for retarding purposes,

Fig. 2.



CHANGES IN MEAN PRESSURE AND HORSE-POWER RESULTING FROM ACCELERATION AND DECELERATION IN A COMPLETE WIND FROM PIT-BOTTOM TO SURFACE.

and produces what is really a negative diagram ; and for the purpose of making this clearer I have shown it below the datum line. From the diagram it will be seen that a steam-pressure of about 50 lbs. per square inch is put against the engine, representing about 600 HP. of braking effect. This method is not, of course, the most economical way of lifting the load from a steam-consumption point of view, but it is in common use because of the importance of raising in a limited time as large a quantity of coal as possible, to avoid the inevitable

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congestion at the bottom of the pit caused by uneven deliveries of coal from the various working-faces.

In this pit 3,200 men and 3,500 tons of coal are wound every 24 hours, and the running of the cage, owing to wire-rope guides, instead of the old type of wooden guide, is as steady as that of a train running at 60 miles per hour on good permanent way—in fact steadier than the running on some railways I could mention. These loads of men and coal are dealt with day after day with hardly ever an accident or breakdown, although the conditions, as to dust above and below ground and dampness in the shaft, are very unfavourable. I am told that statistics show that the fatal accidents arising out of winding men in this country are only 1 in 40,000,000. Such results certainly reflect great credit on our mining engineering colleagues. This is an interesting subject, but owing to the limited time for my Address I must not go further into detail.

Before leaving the subject I should like to add a few words of a personal character. Serious accidents from causes entirely beyond the control of the mining engineer do happen in collieries—falls of roof, sudden outbursts of gas and water—and there is still considerable loss of life from these causes. The heroism of the mining engineer, and indeed of all the workmen, at these tragic times, is well known and honoured by all. I have been down collieries twice under such circumstances, and have been filled with admiration for the calm and dignified courage shown by all, there being only the one desire, to take every possible means to save the lives of those in danger and ignore the personal risks which at such times are inevitable.

Training.

Owing to The Institution embracing so many classes of engineers, the course of training leading to membership must be of a broad nature. Since Sir William White's Committee drew up its scheme, the Council has constantly given consideration to the question, and modifications in our requirements are from time to time adopted.

I am aware that this question is rather controversial, and any views I express must be regarded as entirely personal opinions.

In speaking on this subject I have in mind men who are anxious to equip themselves with suitable training for taking leading positions in our profession, or in the even wider fields of industrial administration. I believe that a general education for engineers, while embracing the essential subjects of an English education, should be based on classics and history as fundamental subjects, for they will broaden the mind and give a boy an interest in subjects

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outside his technical work and thereby help him in after life when he is meeting men of other professions and facing problems requiring the broader training. If he leaves school with a widely trained mind and without any previous attempt at specialization, he will, I believe, all the more easily make progress with his technical subjects. I should like our Students' Examination to be more a test of high level of general education, with little reference to technical subjects. I feel that the Associate Membership Examination is the real test of their fitness for membership and that admission to Studentship should be open to all boys who satisfy us that they have reached a high level of general education. The Oxford and Cambridge Higher Certificate Examination represents what I mean—an examination within reach of boys only of exceptional ability, who are likely to make good in any walk of life. I favour accepting such qualifications as admitting to Studentship without ascertaining what aptitude the candidates may have for technical subjects at this early age.

Our qualifications have to include both practical and technical training, and different views are held as to the desirable sequence of these two subjects. Expediency on the one hand, and opportunity on the other, however, step in and rather tend to decide the matter. The particular point affects men taking mechanical engineering mainly, though not entirely.

I assume technical training will be represented by a course at some University, and that practical training will be obtained in a works. Which is to be taken first? The University Professor will quite rightly say: "Give me the boy direct from school, before he has thrown his books aside and while his mind is still in a receptive condition for taking up new subjects. With reasonable application he will take his qualifying examination easily in his first year and honours in his third or fourth year." I agree with him; but then we find we are up against a difficulty, as I will explain. The student leaves the University already a man, and there is great difficulty in his obtaining practical training in a works, because of his age. Practical training is desirable, not only to familiarize ourselves in the use and output of various tools, the progress of manufacture, and the various uses of machinery, but also to enable us to understand the psychology of the workmen. This is more easily absorbed by the boy going directly from school into the works, when, owing to his age, he is really an apprentice. The workmen teach him willingly, and talk freely with him about their work and their domestic life, and such knowledge is invaluable to the boy in after life when he is in a position of authority over large bodies of workmen. In the workshop a boy will meet men of very fine, and some of rough,

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character, but the experience thus gained will be of the greatest value to him in after life. He is able to understand their point of view, and many troubles are avoided because of this fact. May I reverse this position. Many disputes are caused from lack of sympathy and understanding of the difficulties underlying the question. On principle I lean to some practical training being taken directly after school life, say at 18, when a boy has had an opportunity of reaching a good position at a public school, and as I have said earlier, expediency points to this practice as the more desirable, especially for boys who intend to become mechanical engineers. I regret that from time to time we have to refuse membership because the applicants have failed to obtain the necessary practical training which they have aimed at after their University course, whereas they would most probably not have had this difficulty had they taken it directly from school and subsequently gone forward to a University. I favour maintaining our Associate Membership examination at the present high level, but the increase in technical and scientific knowledge is so great that, in view of our Institution embracing so many branches of engineering, the papers must contain questions covering a wide range of technical subjects. The object of such an examination should be to ascertain how much a candidate knows, rather than to prove, by setting too difficult questions, how limited and specialized his knowledge may be.

Standardization and Specifications.

Sir John Wolfe Barry's name will live long in the minds of engineers, not only for his professional work, but as the man who had the courage and energy in his late years to inaugurate the British Engineering Standards Association, now carried on with the assistance of many eminent engineers under the able Chairmanship of our Vice-President, Sir Archibald Denny. It has been my privilege to be his colleague in this work for several years, and, although I rejoice at the progress made, I am conscious there is still much to be done. Sir John Wolfe Barry brought the subject before the Council of our Institution in 1901, and from the small Committee then formed of eight Members, the organization has grown enormously, and now has 475 Committees and 2,300 Members. Engineers have shown great public spirit in giving time and attention to this work, entirely on an honorary basis, proving, I think, the great importance they attach to the development of the work. The engineering and steel firms throughout the country have generously supported these efforts by contributions to the funds, but this is a severe tax on industry in the present state of things, and it

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is earnestly hoped that some measure of Government help may be forthcoming, so as to enable the Association more fully to meet the requirements of the country. The Association, however, does not believe in waiting for this, and is itself widening both the basis of its work and its method of appeal to the industries of the country. There can be little doubt that the more rapid adoption of standard specifications as understood by the Association would do much towards helping industry at the present time, and this alone would justify any Government in giving prompt and adequate support. In addition to the preparation and issue of British Standard specifications dealing with great industries of the country, substantial aid has been given to our export trade by translating and disseminating in large numbers the principal specifications, for use in foreign countries. This work has lately to a considerable extent had to be abandoned, owing to lack of funds, but special efforts are now being made to revive it, in view of the very important bearing it has on our export trade. Already great progress has been made in the preparation and issue of specifications dealing with general engineering products, and especially in electrical matters, leading to considerable simplification and substantial economy; but, when we come to consider the broader aspects of standardization, the value of this great co-ordinating authority is apparent, and it has proved itself to be a necessity in the interests of the public. In many areas there exist differences of voltage which prevent direct communication between the various towns in the same area and lessen the possibility of providing electric power economically from one large central station. Further, the electrification of the various main lines, and the grouping of the railways, are factors which emphasize the necessity for uniform practice. In Australia and Africa the presence of three gauges is causing difficulty in communication between different areas. These difficulties point to the desirability of some co-ordinating authority, and although to a great extent we deprecate Government interference, the convenience of the public should be the first consideration on the part of engineers, and general rules for uniformity, arrived at by a Government Department after consultation with the leading engineers on the technical requirements, should be accepted and adopted by the engineers in charge of the different railway- and tramway-systems, even though they may not embody precisely their own views.

A new section of the British Engineering Standards Association has just commenced to deal with colliery specifications, where there is much variation in design of an entirely unnecessary nature.

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Many of the differences are merely those of detail dimensions, embodying no question of engineering importance, but the lack of uniformity hinders the introduction of any change, owing to having to consider each case on its merits, and means unnecessary expenditure on drawings and patterns. Further, the introduction of an article of new design on American and German railway- or tramway-systems is much easier than on a British railway, not because of unwillingness on the part of British railway engineers, but because of lack of uniformity of design in use on each railway or tramway. I would urge that, where no principle or question of safety is involved, a spirit of compromise would do much to assist the adoption of new productions which in many cases have already proved their merits in service.

The introduction of alloy steels for use in engineering has opened up new possibilities, and we have not yet reached the limit of strength for a given weight of material. This is a matter of such interest to engineers designing bridge-work and machinery that I must enlarge a little on the question. The metallurgy of steel is a new science only recently developed, and I attach great importance to close co-operation between the engineer and the metallurgist in drafting the specifications, embodying on the one hand the requirements of the engineer, and, on the other, the possibilities of the steel-maker. The higher horse-powers and steam-pressures, previously referred to, which are now required for marine and electrical engineering, have brought with them practical manufacturing difficulties that require careful study, and suitable modifications in design can only be arrived at by close consultation between the metallurgist and the engineer. The best instance I know of this co-operation is illustrated by the high level of efficiency which the Admiralty has achieved and the satisfactory immunity from failures and breakdowns which has resulted. I believe this to be largely attributable to their practice of consulting the works engineers and metallurgists who have the work to produce, and agreeing on a specification which meets their requirements and at the same time avoids the introduction of unnecessary manufacturing difficulties.

Before concluding, I wish to say a few words on the position of engineering *vis-à-vis* the public. It is our duty and privilege to cater for the comfort and health of our fellow citizens. In transport by land or sea or air they depend on the good work of the engineer for their safety and their comfort. The supply of water and the disposal of sewage is entrusted to us. Inefficient harbours not admitting vessels of sufficient tonnage and draught at one cause

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inconvenience and suffering to our fellow citizens. No other profession, I imagine, is so directly responsible for all these domestic comforts. These are, I feel, reasons why we must strive to attain efficiency in our training and in our daily work, realizing that our privileges and opportunities carry with them great responsibilities, the full recognition of which will enable each of us to add to some little extent to the high reputation which engineering as a profession has already attained.

Sir ROBERT ELLIOTT-COOPER, K.C.B., Past-President, said it was with the greatest pleasure that he had the privilege that evening of moving :—

> "That the best thanks of The Institution be accorded to the President for his Address, and that he be asked to permit it to be printed in the Minutes of Proceedings."

The President had dealt with some of the most important subjects that it was possible to imagine. He did not suppose that there was a single member present that evening who was not personally interested, especially in the first of the four subjects into which the Address was divided, namely, the steel industry. All engineers no matter what their particular branch of work might be, knew that in some form the production and manufacture of steel must force itself upon their attention. Colliery engineering, at a time like the present, when Commissions of inquiry into the various questions connected with coal-production and the production of iron and steel were sitting, was, indeed, a subject not only for The Institution but for the country at large to consider ; and it was good that men like the President should be able to show with authority to what extent the prosperity—in fact, almost the whole commercial life —of the country was dependent on those matters. The Institution would not have arrived at the pre-eminence it had reached were it not for its efforts in the cause of engineering training. When he (Sir Robert Elliott-Cooper) was on the Council he took a great interest in that subject ; he had had many young men through his hands, and he knew what a great work The Institution had done in furthering and fostering the education and training of the young engineer. It was a matter of some little interest to think that the President and he were educated—although at an interval of a good many years—in the same town. They both received their professional training in Leeds. Engineers would fail to achieve all their aims in the other fields referred to by the President if it were not for the great work of standardization—largely due, as the President had said, to the late Sir John Wolfe Barry. He had been on different

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Committees of the British Engineering Standards Association, and he knew what an immense advantage its work had been, not only to engineers, but also and in a still greater degree to those who had to produce articles which were embraced within the scheme of the Association. He could remember the time when an engineer was proud of designing a bridge to be constructed of angles, tees, etc., of sections that had never been thought of before. The unnecessary labour and expense incurred by the manufacturers in meeting those whims and fancies could be imagined; and nothing whatever was gained by such procedure. That was a matter of the past. The manufacturers now produced practically everything that could reasonably be required by an engineer. He was sure that the President, in bringing the subject so well before the meeting, had done a great service not only to those present, but also to those who would read his Address in many distant lands.

Mr. E. W. MONKHOUSE seconded the motion.

The resolution having been passed by acclamation,

The PRESIDENT said he did not suppose anyone in the room had envied him his task that evening, and he appreciated the consideration with which they had listened to his Address. He had been anxious, in speaking to The Institution, to avoid confining himself to a subject which would be interesting only to certain classes of members. He had wished to make the Address widely interesting, knowing that his audience would be representative of different kinds of engineering. In the limited time he had allowed himself, even to touch on one question meant being very concise, but it was much more the case when he had to deal with four. It would have been a greater pleasure to him had he felt free to go rather farther into any one of those subjects. He wanted to assure the members that he realized what was involved in the office of President of The Institution—the greatest honour that could come to anyone in the profession. It came after years of careful work on the Council and as a Vice-President, so that he could not pretend not to know what was involved, and he asked the members for their loyal support during the coming year.

The PRESIDENT then presented the Telford, Watt, George Stephenson, and James Forrest Medals awarded by the Council for Session 1924–25, and the other awards in respect of Papers read during that Session, as well as those for Selected Engineering Papers and for Students' Papers, were announced (pp. 18–20).

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MEDALS AND PREMIUMS AWARDED

SESSION 1924–1925.

FOR PAPERS READ AND DISCUSSED AT THE ORDINARY MEETINGS.

1. A Telford Gold Medal to Donald Paterson, M. Inst. C.E., for his Paper on "The Johore Causeway."

2. A Watt Gold Medal to Ernest Hinkly Salmon, D.Sc. (Engineering), M. Inst. C.E., for his Paper on "The Southampton Floating Dock."

3. A George Stephenson Gold Medal to Leopold Halliday Savile,[1] C.B., M. Inst. C.E., for his Paper on "The Demolition of the Harbour and Defence Works of Heligoland."

4. A Telford Premium to George Mitchell, M. Inst. C.E., for his Paper on "The Water-Supply of the City of Aberdeen."

5. A Telford Premium to Thomas Ernest Stanton,[2] C.B.E., D.Sc., F.R.S., M. Inst. C.E., for his Paper on "The Measurement of the Pressure of the Wind on Structures."

6. A Telford Premium to Francis Ernest Wentworth-Sheilds,[3] O.B.E., M. Inst. C.E., for his Paper on "Southampton Floating Dock: Subsidiary Works."

7. A Crampton Prize to Professor Arnold Hartley Gibson,[1] D.Sc., M. Inst. C.E., for his Paper on "The Investigation of the Surge-Tank Problem by Model Experiments."

8. A Manby Premium to Philip Warwick Robson, O.B.E., M. Inst. C.E., for his Paper on "The Large Water-Tube Boiler."

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[1] Has previously received a Crampton Prize.

[2] Has previously received a Telford Gold Medal, a Howard Quinquennial Prize, a George Stephenson Gold Medal, and two Telford Premiums.

[3] Has previously received a George Stephenson Gold Medal.

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FOR SELECTED ENGINEERING PAPERS.

(Being Original Communications ordered by the Council to be published without Discussion.)

1. A Telford Gold Medal and the Indian Premium to Sheldon Arthur Steward Bunting, M.B.E., M.A., B.Sc., M. Inst. C.E., for his Paper on "The Development Works at Sambhar Salt Lake, Rajputana, India."
2. A Telford Gold Medal to Andrew Robertson, D.Sc., Assoc. M. Inst. C.E., for his Paper on "The Strength of Struts."
3. A Telford Premium to Andrew Don Swan, M. Inst. C.E., for his Paper on "Vancouver Harbour, B.C. (Canada)."
4. A Telford Premium to Clement Henry Cruttwell, Assoc. M. Inst. C.E., for his Paper on "Construction of a New Landing-Stage and other Works at Gosport."
5. A Telford Premium to John Williams McLaren, B.Sc., M. Inst. C.E., for his Paper on "Interlocking Steel Sheet Piling for Temporary Dams at Grimsby."
6. A Telford Premium to John Lawrence Hodgson,[1] B.Sc., Assoc. M. Inst. C.E., for his Paper on "The Orifice as a Basis of Flow-Measurement."
7. A Webb Prize to Arnold Robert Johnson for his Paper on "The Penang Hills Railway."

FOR PAPERS READ AT STUDENTS' MEETINGS IN LONDON AND BY STUDENTS BEFORE MEETINGS OF LOCAL ASSOCIATIONS.

1. A Miller Prize and the James Forrest Medal to Edmund Jule Rang, B.Sc., Stud. Inst. C.E., for his Paper on "Some Properties of Mild Steel, with special reference to its Behaviour at High Temperatures."
2. A Miller Prize to David Lloyd, M.Eng., Assoc. M. Inst. C.E., for his Paper on "Liverpool Corporation Waterworks, Vyrnwy Supply: Construction of Filter Beds Nos. 14–16, and the Sand-Washing Depot at Oswestry, 1923–1924."
3. A Miller Prize to Harold Frederick Lea, Stud. Inst. C.E., for his Paper on "Arterial Road Construction."
4. A Miller Prize to Hector Archibald Macnab, B.Sc., Stud. Inst. C.E., for his Paper on "Paris Metropolitan Railway."

[1] Has previously received a Telford Premium.

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5. A Miller Prize to Thomas William Marsh, B.Sc. (Engineering), Stud. Inst. C.E., for his Paper on "The Littoral Drift of Shingle."

6. A Miller Prize to Charles Richard Smith, B.Sc., Stud. Inst. C.E., for his Paper on "The All-Concrete Road."

7. A Miller Prize to William Cuthbert Knill, B.Sc., Stud. Inst. C.E., for his Paper on "Subsidence due to Coal Workings and its effect on the Surface and the Structures thereon."

Bayliss Prizes.

Bayliss Prizes, awarded on the results of the October and April Examinations, 1924–1925, respectively, to Alfred Edward Tuffley, Assoc. M. Inst. C.E., and Eric Olding Rose, B.Sc. (Engineering), Stud. Inst. C.E.

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17 November, 1925.

Sir WILLIAM HENRY ELLIS, G.B.E., D.Eng., President,

in the Chair.

The PRESIDENT announced that the Council had elected Sir Brodie Henderson as a Vice-President, to fill the vacancy caused by the death of Captain Sankey.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

(*Paper No. 4565.*)

"The Mechanical Properties of Steel."

By Professor WILLIAM ERNEST DALBY, M.A., B.Sc., F.R.S., M. Inst. C.E.

FOR many years past the Author has been engaged in research on the strength of materials, and his object in this Paper is to place before The Institution some of the results of practical interest.

As a result of continued experimenting with a view to record automatically the process of testing a specimen of metal, four instruments have now been standardized, namely: (1) the "no-load to break" recorder[1]; (2) the elastic recorder for tensile tests[2]; (3) the elastic recorder for tensile and compression tests[3]; and (4) the torque-twist, or torsion, recorder.[3]

The recorders furnish a photographic negative on which the progress of the test is traced by a spot of light, and they are, therefore, free from errors of pencil friction and inertia, and from the personal equation. In the no-load to break recorder the extension of the gauge-length is magnified on the negative three times. The elastic recorder traces the history of the first 0·03 inch extension, magnified 150 times. This magnification requires a mechanical construction which differs entirely from the extensometer of the breaking recorder. A microscope is fitted to enable the test to be followed and controlled. The push-pull recorder is similar to the elastic apparatus, except that it is designed to record a push as well as a pull. The limits of the records are 0·02 inch in tension and compression. The range of twist in the torque-twist recorder is ± 5 degrees.

[1] Proceedings Roy. Soc., Series A, vol. lxxxvi (1912), p. 414, and vol. lxxxviii (1913), p. 281.

[2] Phil. Trans. Roy. Soc., Series A, vol. ccxxi (1920), p. 117.

[3] Proceedings Roy. Soc., Series A, vol. ciii (1923), p. 8.

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A calibrating grid has been prepared for each of these instruments, consisting of a glass plate marked with scales for load and extension, compression, or twist. By placing the grid over a record the co-ordinates of every point of the curve can be read. These grids are made by direct loading and by direct extensions or twists, so that the spherical error due to a spot tracing its curve on the flat surface of the photographic plate, instead of on the curved surface theoretically required, is automatically corrected when the negatives are read off through their grids.

The recorders are designed on a common principle, although the constructional details differ.[1] A load on the test-piece is measured by the elastic deformation of a steel bar coupled to it, and deformation of the test-piece is measured by an extensometer forming part of the instrument; the two deformations are combined at right angles, magnified optically, and recorded photographically.

Fig. 1.

Fig. 1 illustrates the principle applied to the tensile recorders. The test-piece A is coupled to the steel weigh-bar B. An axial load applied to the pair is carried equally by both. The proportions of the pair are so chosen for the breaking instrument that the applied axial load will break the test-piece well before the stress in the weigh-bar has reached the limit of elasticity. Therefore the extension of B, being always elastic within the range of loading of the test, is a measure of the load on the test-piece A.

The principle of measuring loads by elastic deformations of a steel bar has been used by many experimenters. The earliest user was probably Mr. G. A. Hirn.[2] Sir Alexander Kennedy, Past-President Inst. C.E., and the late Professor A. G. Ashcroft, M. Inst. C.E., have based on this principle instruments for drawing load-extension diagrams. The necessary magnification of the elastic deformation of the weigh-bar was mechanical, and the record was made by a stylus scraping over smoked glass. The novelties in the Author's instruments lie in the combination of optical devices to magnify the deformations and photographic devices to record the diagram, so eliminating errors in the record due to friction, inertia, and the personal equation. In consequence the records can be taken in a few seconds if desired. Each instrument is self-contained. It carries a source of light, the necessary optical elements, and its own camera. The optical

[1] For details see "Strength and Structure of Steel and Other Metals," by W. E. Dalby. London, 1923.

[2] "*Les Pandynamomètres*," Paris, 1876.

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elements are within the hollow weigh-bar and are designed to eliminate the small errors due to the slightest bending of the weigh-bar under a not-quite-axial load. A beam from the source of light—a small 3-volt lamp—passes through a small hole and is reflected successively from two mirrors whose axes are at right angles, the beam finally coming to a focus on the camera screen. An extension of the weigh-bar tilts one of the mirrors, and so causes the spot of light to travel across the plate proportionally to the extension of the weigh-bar. An extension of the gauge-length of the test-piece is carried through an extensometer to the second mirror so that the spot moves lengthwise on the plate proportionally to the extension of the test-piece. Simultaneous movements of the two mirrors cause the spot to trace a path which is the historical record of the test.

The tensile recorders may be used in any machine capable of applying an axial load, although the elastic instrument is better used in a machine specially designed for it. With the assistance of Messrs. Buckton and Company, of Leeds, the Author has designed hydraulic plant specially for use with these recorders; a lever machine is required only for initial calibration, its control being unsuitable for the recorders.

Fig. 2 shows the test-piece A and the weigh-bar B in the form used in the torsion recorder. The proportions of the combination are chosen so that the maximum torque applied to the test-piece produces a shearing stress well within the elastic limit of the torque weigh-bar. Therefore the twist of the torque weigh-bar is a measure of the torque applied to the test-piece.

To aid in the interpretation of the diagrams and for convenience of reference, particulars of the apparatus are tabulated :—

Recorder.	Testing-Machine.	Max. Load.	Max. Extension.
No load to break. .	Hydraulic tensile.	15 tons	2 inches
Elastic tension only .	Hydraulic tensile.	15 tons	0·03 inch
Elastic push-pull. .	Hydraulic push-pull.	± 9 tons	± 0·02 inch
Torsion	Hand application of torque through gearing.	± 2½ inch-tons	± 5 degrees

The Process of a Test.—With these instruments, unlike the usual testing-appliances, the variable which the operator controls in the process of testing is the strain. On the application of a definite

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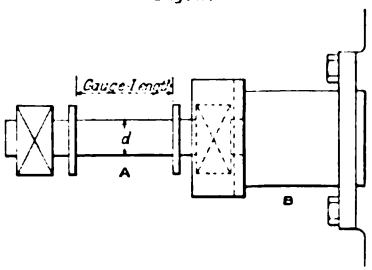
stretch to the test-piece, the corresponding load is recorded automatically. The process of a test may be infinitely varied, but however complex the variation, the history of the process is truly recorded on the photographic negative. For example, stretching may be stopped at any stage; the load may be partially or wholly removed and re-applied; the time of stretching may be varied; and processes may be devised to search for a special property of the metal. The properties of one metal may be probed by the application of a selected succession of varied processes. The properties of different metals may be compared by means of records of the application of one defined process to each metal in turn.

In comparative tests the times occupied in making the tests should be of the same order—although the time-interval can be varied within wide limits without appreciable influence on the records. For example, there is no appreciable difference of form between a continuous record taken in 30 seconds and one taken in 5 minutes from test-bars of the same material and quality. But differences show when the time-interval is reduced to 0·1 second. Creep complicates the record if loading is stopped after the elastic limit has been passed. The stretch or compression increases slowly under constant load, so that the record then becomes a function of the time.

Fig. 2.



Examples of Records.—Three simple processes bring to light the accepted strength constants of a metal. These are a continuous stretch to fracture; a continuous stretch to 0·03 inch recorded on a high scale of magnification; and a continuous twist to 5 degrees. The metal selected for illustration is a mild steel of approximately the following percentage composition :—Carbon, 0·22; manganese, 0·96; sulphur, 0·06; phosphorus, 0·035; silicon, 0·08.

The test-pieces (*Figs. 3* and *4*) were turned from a bar $1\frac{1}{2}$ inch in diameter. The processes applied were :— (1) A continuous stretch to fracture (Fig. 5, Plate 1); (2) a continuous stretch to 0·035 inch (Fig. 6, Plate 1); (3) a continuous twist to 5 degrees (Fig. 7, Plate 1).

From the data recorded in Fig. 5, it will be seen that the peak load previous to plastic deformation is 23·8 tons per square inch and the maximum load 32·9 tons per square inch. Using the area

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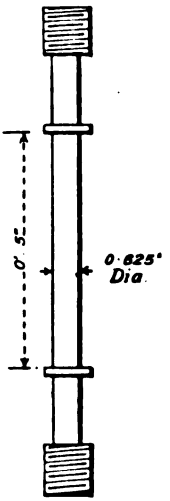
of the bar corresponding to the diameter of the fracture, namely 0·125 square inch, the load at the instant of fracture converts to 56 tons per square inch. The actual extension of the 5-inch gauge-length, 1·45 inch, is equal to 29 per cent., and the reduction of area is 59 per cent.

Fig. 3.

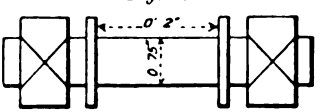


The elastic record (Fig. 6) shows that the load increases proportionally with the stretch up to about 6·25 tons, or 20·4 tons per square inch; and the peak load, namely 7·3 tons, converts to 23·8 tons per square inch. After the fall of the load to 6·35 tons the metal appears to slip at a constant load of 20·7 tons per square inch. The value of the modulus of elasticity, E, is 13,050 tons per square inch.

The torque-twist record (Fig. 7) shows that the twist increases proportionally to the torque up to about 0·9 inch-ton, and then rounds off into a twist at a constant torque of 1·2 inch-ton. The modulus of rigidity, C, represented by the slope of the elastic line, is 4,930 tons per square inch. The shearing stress at the circumference of the test-piece corresponding to the torque of 0·9 inch-ton is 10·86 tons per square inch. Twisting continues, and the breakdown of the material proceeds radially inwards until practically the whole of the section is twisting under a uniformly-distributed shearing stress, which Fig. 7 shows to be 10·86 tons per square inch. The slipping load calculated upon the supposition that a uniform distribution of stress exists over the section substantially agrees with the slipping load calculated upon the law that the stress is proportional to the radius.

The Test-Piece.—The Author was led to the method of defining the gauge-length by means of flanges as a result of many months of experimenting, which proved to him that in the elastic tests the gauge-length should not be defined by any method involving the use of clips held in place by pointed steel screws driven into the test-piece, if during the test these screws have motion relative to the test-piece. Such screws cannot be properly driven into hard material, and in soft material the pits made by the screws elongate as the stretch proceeds, and the points slip. The effect of the flanges on the value of E proved to be negligible, and their effect on the

Fig. 4.



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ultimate extension of soft materials like mild steel was to reduce it, in comparison with the flangeless bar, by about 3 per cent. This is immaterial, however, when all the test-pieces are made to the same proportional dimensions, because then the principle of similarity shows that direct comparison of results can safely be made.[1] Where flanges cannot be turned on the test-piece, detachable flanges are clipped on to it in such a way that there is no relative movement between the points of the screws and the test-piece as the test proceeds.

Records of Varied Processes.

The tests already described are those by which the usual physical constants of a material may be quickly determined with an accuracy sufficient for the practical purposes of the engineer. The testing processes, however, may be varied in many ways, some of which will now be described.

(i) *Looping in Tension.*—This process may be defined thus: Stretch the test-piece to an assigned limit; remove the load; re-apply the load and stretch to a second assigned limit; remove the load; and repeat this as often as may be desired. A record of this process with the no-load to break instrument is shown in Fig. 8, Plate 1. During the stretch the load was reduced to zero and re-applied eight times. All reapplications after the first show a slight increase in the reapplied load before stretching continues. The record of a removal and a reapplication appears to be a straight line approximately parallel to the elastic line through the origin, the slope of which can just be distinguished. Even just before local slipping is proceeding to fracture, a removal and reapplication is met with an elastic response, as indicated by the line L L. Only when the reapplication reaches slipping value does the stretch to fracture continue.

This elastic response is not a property peculiar to the mild steel tested. Fig. 9, Plate 1, is the record of a similar test on 0·8-per-cent. carbon steel. Comparison of the records shows that slipping continues in the high-carbon steel when the reapplied load reaches the value of the slipping load, the small reapplication peaks being absent. The record of the process is seen to be an incipient loop. The extension scale of the no-load to break instrument with which the diagram was taken is too small to allow inference about elastic

[1] Particulars of these tests are given in a contribution by the Author, "A Standard Form of Test-Piece," Report of the British Association, Toronto, 1924, p. 322.

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property. All that the diagrams tell—and that is remarkable enough—is that when stretch is stopped the metal is not plastic like putty but possesses elastic property which is at once exhibited by a reduction of load. When the load is reapplied, the metal stretches elastically until the slipping load is reached, and then its behaviour changes suddenly. All appearance of elastic property vanishes, and stretch continues putty-fashion.

Fig. 10, Plate 1, is the record of a test made by the elastic instrument on mild steel of approximately the same quality as that of Fig. 8. This diagram shows that neither the removal nor the reapplication of the load is strictly an elastic process. The record encloses a loop. The area of the loop represents the work done on the internal crystalline structure of the metal during the removal and reapplication of the load. The area of the loop is small; but, as the carbon-content increases, the loop-area increases. Fig. 11, Plate 1, illustrates this. It will be noticed that a curve has been traced from the origin. The inference is that the test-piece has already been overstrained; and this inference is correct. The piece had been stretched from a primitive gauge-length of 5 inches to 5·03 inches before the test recorded in Fig. 11 was made.

A comparison of the looping-in-tension test on different metals shows that each has its own characteristic series of loops defined by the area of the first loop and the rate of increase of area of successive loops. The rate of increase of loop-area decreases as stretching proceeds, tending to a maximum and constant value.[1]

(ii) *Application of Unlimited Strain in Tension or Compression after an Application of Stress of Opposite Kind within the Elastic Limit.*—Fig. 12, Plate 1, shows the record of an unlimited stretch after the application and removal of a compressive load within the elastic limit. Fig. 13, Plate 1, is the record of an unlimited compression after applying and removing a tensile stress within the elastic limit. A comparison of the two records demonstrates substantially that :—

(1) The value of E is the same in tension as in compression;

(2) Breakdown occurs at nearly equal values of the load, namely, 10·3 tons;

(3) The manner of breakdown is the same in tension as in compression.

The equality of E in tension and in compression is shown by

[1] Full particulars of this work, together with illustrations of the characteristic loop series of different metals, have already been published by the Author, "Researches on the Elastic Properties and the Plastic Extension of Metals," Phil. Trans. Roy. Soc., Series A, vol. ccxxi (1920), p. 117.

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printing the two curves of Figs. 12 and 13 together, thus giving the combined diagram of Fig. 14, Plate 1. The similarity of the manner of breakdown is shown by printing the curves together, reversed, in the way shown in Fig. 15, Plate 1.

No value of the elastic modulus is given, because the Author prefers to determine this from a longer gauge-length than 2 inches. This short gauge-length is necessary when making compression tests, to avoid lateral bending.

(iii) *Alternating Tension and Compression between Positive and Negative Limits of Strain.*—Fig. 16, Plate 1, shows a compression from zero to 0·016 inch followed by a removal of load and the application of a tensile load to stretch the gauge-length to 0·015 inch. The tensile load is then removed and reversed to compression, and the gauge-length is compressed to its primitive length of 2 inches. The load is then removed, leaving a permanent stretch of about 0·003 inch.

This curve records: (1) An extremely rapid breakdown in compression; (2) an elastic response to the removal of the compressive and tensile loads; (3) a rapid change to plastic yield at the reversal of compression to tension and vice versa; and (4) a shape which is characteristic, and is called by the Author the "plastic contour" of the metal. This diagram shows also that when either a compressive load or a tensile load is removed, the response of the metal is quasi-elastic.

(iv) *Alternating Tension and Compression between Widening Positive and Negative Limits of Strain.*—Fig. 17, Plate 2, is a record of this process carried out on a piece of 0·8-per-cent. carbon steel. Starting from the origin, compression was applied to shorten the gauge-length by 0·007 inch. The compression was removed, and thereby the shortening was reduced to 0·0032 inch. The gauge-length was next stretched to 0·0068 inch. The tensile load was then reduced to zero, and the gauge-length shrank elastically to 0·002 inch. Changing to compression, the gauge-length was shortened by 0·0137 inch. The alternation of load was continued with increasingly wide limits of strain as shown on the record. The final process was compression to the primitive gauge-length and removal of the load, leaving a permanent stretch of 0·005 inch.

The envelope of the diagram—the plastic contour of the metal—is substantially of the same shape as it would have been if one alternation of load had been applied to stretch and compress the test-piece between the widest limits of strain shown on the record. The diagram is additional evidence of elastic response to reduction

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of tension or compression, and plastic response to change from tension to compression or from compression to tension.

(v) *Looping in Tension and in Compression between equal Positive and Negative Limits of Strain.*—Fig. 18, Plate 2, is the record of this process applied to a brass test-piece. Starting from the origin, the test-piece was compressed 0·01 inch. The metal broke down at 4·15 tons and stretched at nearly constant load until at A the load was removed and reapplied, and the compression continued to B. The removal and reapplication produced the loop A. The load was removed at B and reversed to tension at C. During the process of the stretch, the three loops were formed at D, E, and F. Tension was removed at G, and a reversal to compression took place at H. The drawing of the loop I, the final removal of the load at J, and the no-load point K complete the diagram. The result of the process was a permanent stretch of the gauge-length of amount O K. This diagram shows that looping takes place equally in tension and in compression; and loops A and F indicate that the loops described from equal loads, whether tensile or compressive, are equal in area. Loops I and D further illustrate this approximate rule.

(vi) *Application of Positive and Negative Torques within Equal Positive and Negative Limits of Twist.*—This process is illustrated by Fig. 19, Plate 2, applied to a brass test-piece $\frac{3}{4}$ inch in diameter and 2 inches gauge-length, cut from a brass rod $1\frac{1}{2}$ inch in diameter, the limits of twist being $\pm$ 4 degrees.

Starting from the origin O, the torque is proportional to the twist up to about 0·5 inch-ton, and then the elastic resistance gradually merges into the plastic, and the material twists at nearly constant torque to 4 degrees at A. The twist is here reversed to 4 degrees negative, as shown by the point B. Twist is again reversed to C, thus bringing the test-piece back to its primitive configuration, after which the torque is reduced and the experiment is ended at D. Successive reversals of twist produce a lenticular area, the boundaries of which are named by the Author the "plastic contour" of the metal in torsion. This contour has a characteristic shape for a metal, however the process of describing it be varied.

(vii) *Application of Positive and Negative Torques between Widening Limits of Twist.*—This process was applied to the brass test-piece used for the test of Fig. 19, and the record is seen in Fig. 20, Plate 2. It may be noted that the advantage of ending a test on the centre of the scale seen in the microscope is that when the plate is developed the last point C should appear on the vertical axis, and thus check the continued accuracy of the apparatus. The various lenticular

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areas AB, A_1B_1, A_2B_2, of the widening limits are seen to fit in accurately with the ± 4 degrees plastic contour; and, further, the ± 4 degrees contour of Fig. 20 coincides with the contour of Fig. 19, omitting, of course, the irregularity of contour of the first test at the elastic breakdown.

(viii) *Looping in Positive and Negative Torques between Equal Positive and Negative Limits of Twist.*—A record of this process is seen in Fig. 21, Plate 2. Starting at the origin O, the piece is twisted about 2 degrees; then the twist is reversed to reduce the torque to zero, and immediately reapplied to twist the piece to A_1. This reversal and reapplication results in a loop on the record. The looping process is repeated at A_1, A_2, B, B_1, B_2, C_1, C_2, and the test ends at C_2.

The plastic contour of this diagram between + 4 and − 4 degrees is practically the same as in the tests of Figs. 19 and 20. This further emphasizes the conclusion that variation of process makes little difference to the shape of the plastic contour between the same limits when the process is carried out continuously.

The Symmetry of the Plastic Contour.

An outstanding deduction from the torque-twist diagram is that the plastic contour of a metal has a characteristic shape between given limits of twist. Further, the characteristic shape is maintained during repeated twists, but after each twist the hardness is slightly increased with all metals except copper, which shows a slight softening. Figs. 22 to 27, Plate 2, illustrate these points. Fig. 22 is a record of twelve twists of mild steel between the limits + 4 and − 4 degrees. The gradually-increasing area of the contour indicates the gradual hardening. After the first twist, succeeding twists produce less hardening. In fact the hardening process tends to a maximum. In Fig. 23 (stainless steel) the limits were widened. The process recorded is: three twists between limits ± 2 degrees; three twists between limits ± 3 degrees; and six twists between limits ± 4 degrees. Fig. 24 (0·8-per-cent. carbon steel) records thirty twists between the limits ± 4 degrees.

In Fig. 25 (brass), twelve twists between limits ± 5 degrees, two well-marked nodes appear where the curves cross. This shows that the twist is the same for the same torque at a node and is independent of the number of repetitions of the twist. With hard copper, Fig. 26, the successive areas of the plastic contours decrease, and two definite nodes are formed. The shape of Fig. 27 (cast iron) is characteristic; the rate of increase of area is small, and the test-piece broke just before the tenth twist.

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In many records the plastic contour is symmetrical about the origin. Absence of symmetry may be due to many causes, which are being investigated. One definite practical conclusion, however, may be stated. If there is want of symmetry, it follows that the positive and negative yield torques at the elastic limits differ. Therefore the conclusion is that the elastic limit of the material in its primitive state should be determined from the elastic range defined by the dimensions of the plastic contour and not from a single experiment with torque of one sign.

Comparison of Records.—Proceeding now to the comparison of torque-twist and compressive-tensile, or push-pull, diagrams, and taking as examples Figs. 18 and 21, both from the same material, it is hard to distinguish the torque-twist process from the push-pull process, so far as the shapes of the diagrams and the loops are concerned. They exhibit the same characteristics. The break-down in each is similar, except where the elastic yield passes into the plastic yield. In the torsional test-piece yield takes place first in the outer cylindrical lamina and proceeds inwards to the centre. This march of the yield from circumference to centre is reflected in the gradual rounding of the corner. Again, the plastic contour of the push-pull diagram is similar to the plastic contour of the torque-twist diagram, as will be seen by comparison of Fig. 20 with Fig. 17, Plate 2, and of Fig. 19, Plate 2, with Fig. 16, Plate 1. In short, the records lead to the conclusions that the metal fails by slipping, however the external load may be applied, that the metal retains its elastic quality during and after slip, and that this elastic quality is exhibited directly the load producing slipping is reduced.

Slip.

The conclusion that failure is due to slip is verified by the microscope. When an etched and polished surface of a metal is examined under the microscope, the test being arranged so that the test-piece can be strained while the surface is under observation, at first, under no strain, the surface appears to be smooth, and only the boundaries of the crystal blocks forming the structure are seen. When strain is applied, the smooth surfaces of the blocks show fine lines running across them in parallel bands. These lines are the stepped edges of the crystal laminæ slipping over one another like a pack of cards, the view in the microscope being the view of the end of the pack.[1]

[1] Slip bands were first observed by Sir Alfred Ewing and Dr. Walter Rosenhain, who described them in the Bakerian Lecture of 1899, Phil. Trans. Roy. Soc., Series A, vol. cxciii, p. 353. Microphotographs illustrating this are shown in the Author's book, "Strength and Structure of Steel and Other Metals," p. 152.

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The crystalline axes of a block are not parallel, but point in all directions. According to one school of thought the blocks are cemented together by amorphous material, that is, the liquid matter out of which the crystals have been formed but which has not been incorporated in the crystal blocks themselves. The crystals, growing in all directions from their arbitrary starting-points in the liquid, meet, and these meeting surfaces are defined by what appears to be a layer of unincorporated material between them. A later school of thought considers that the matter between the blocks is itself crystalline, but that each crystal is of a higher order of minuteness than the blocks, the crystal blocks of gross size being cemented together by crystal fragments of minute size.

The study of the process of slipping is complicated by the varying inclination of the crystalline axes and the influence of the boundaries between the crystal blocks. The researches of Professor H. C. H. Carpenter and Miss C. F. Elam, however, enable the influence of the boundaries to be discarded, because they have succeeded in producing single crystals of aluminium large enough to form test-pieces.[1] A study of the processes of failure can therefore be carried out on a single-crystal test-piece, thus eliminating the influence of the boundaries. Mechanical tests made on test-pieces machined from a single crystal give substantially the same form of load-extension diagram as the multi-crystal test-pieces; and microscopic examination reveals the same phenomenon of slip bands as seen in the multi-crystal test-pieces. The single-crystal test-piece contributes further evidence of failure through slipping, by the manner in which the test-piece breaks. A round test-piece of mild steel breaks usually with a cup-and-cone fracture; but a single-crystal test-piece breaks with an oblique plane fracture. Therefore it may be concluded that the cup-and-cone fracture is a consequence of the multi-crystal structure.

The inference from the mass of work accumulated on the subject, and from the recent remarkable experiments and deductions of Professor Carpenter, Professor G. I. Taylor, and Miss Elam, is that failure of a metal takes place by the slipping of the crystal blocks within themselves.

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[1] The Bakerian Lecture of 1923, by Professor Taylor and Miss Elam, was devoted to the discussion of the slip-band systems in a single-crystal aluminium test-piece, identified by X-ray analysis. In 1925 Professor Carpenter and Miss Elam gave the results of their recent work on the distortion of single-crystal test-pieces under load. Proceedings Roy. Soc., Series A, vol. cvii, p. 171.

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The Space Lattice.

Failure by slipping, however, gives no explanation of the remarkable phenomenon observed in all the records of load-strain or torque-twist already discussed, namely, that after failure by slipping, reduction of load is met by an elastic response. Consider the loops Fig. 28, Plate 2, and Figs. 29 and 30, Plate 3. The two loops in Fig. 28 were taken after the test-piece had been stretched 8 per cent.; the single loop in Fig. 29 after 18 per cent. stretch; and the single loop in Fig. 30 after 22 per cent. stretch. The test-bar was on the point of fracture after the stretch of 22 per cent., a waist having formed 1½ inch above the lower flange. The loop (Fig. 30) differs only in size from those during the extension. It is larger than the loop at 8 per cent. stretch, but slightly smaller than the loop at 18 per cent. stretch—corresponding to the maximum load carried by the piece. This establishes that the stretched metal possesses elastic property even when it is on the point of fracture.

Light is thrown on this question by recent researches with X-ray analysis. Professor Sir William Bragg has established that a crystal is composed of material points distributed in a geometrical pattern through the volume of the crystal, forming what is called a "space lattice." It has been established by Dr. Arne Westgren and Mr. Gösta Phragmén[1] that the space lattice for alpha iron is a "body centred cube." This means that the element of the pattern, or the "repeat," is a cube with a material point at each corner and one material point at the intersection of the cube diagonals. This centred-cube pattern repeats itself through the volume of a single crystal, just as a wall-paper design repeats itself over the wall surface. Further, it has been determined that the side of this cube measures about one hundred millionth of an inch (2·87 AU) at ordinary temperatures.[2] These workers have further established that the space lattices of beta iron and delta iron are similar to that of alpha iron, but that the space lattice of gamma iron is "face centred," which means that there is a material point at each corner of the cube and a point at the intersection of the diagonals of each of the six faces. The gamma-iron lattice of the austenitic steels is enlarged by dissolved carbon. The lattice for pearlite is complex and has not yet been fully established.

Whatever be the actual pattern of the different varieties of inner structure, the simple body-centred cube may be depended upon for

[1] "X-Ray Studies on the Crystal Structure of Steel." Journal Iron and Steel Institute, vol. cv (1922), p. 241.

[2] The Ångström unit = 10^{-10} metre.

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some general ideas which will help in the understanding of the problem of elastic response to reduction of load after slip has taken place. It must be imagined that the material points of the space lattice are kept locked in their geometrical pattern by some cause which has the total effect of mutual attractions between the material points. It is further imagined that stress distorts the lattice and then causes slip along a plane within the lattice. When the stress is removed, the lattice re-meshes, and the geometrical pattern is restored. It is true that the material points ranged along one side of the slip-plane face a new set of points on the other side, but this change of partners has been effected without permanently changing the geometrical pattern of the space lattice, except that in the immediate neighbourhood of the slip-plane minute crystals have been rubbed into individuality from the main lattice. That some work is done is shown by the experimental fact that slip is accompanied by evolution of heat. This question has been recently studied by Professor Taylor and Mr. W. S. Farren,[1] working on a single crystal of aluminium.

Consider now a multi-crystal structure, which may be regarded as an aggregation of space lattices, each lattice with a different orientation. It is possible that in each crystal block the slip-plane bears some angular relation to the crystalline axes, and as these axes point in all directions, the slip-planes may be expected to do the same, which conclusion is borne out by microscopic examination. A plastic extension may, therefore, be regarded as the integral of innumerable minute slips in the space lattices of the crystal blocks of the structure, and an elastic deformation as the distortion of the space lattices without slip.

The First Slip.—To consider the conditions in which slipping takes place, imagine an external load of any type producing within the material of the test-piece lines of stress, amongst which are lines of maximum shearing stress. Along any one line of stress the blocks present a varied orientation, which places their respective space lattices in different angular positions relative to the direction of stress. Therefore some are in favourable and some are in unfavourable positions for resisting slipping. The total resistance to slipping along an element of a line of stress is the statistical summation of the resistances of all the space lattices through which the element of the line passes. The resistance along equal elements of a line of stress cannot be identical, because the blocks vary in

[1] "The Heat Developed during Plastic Extension of Metals." Proceedings Roy. Soc., Series A, vol. cvii (1924), p. 422.

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size and the number of space lattices contributing their resistances along the element is therefore indeterminate. But, of all possible directions of slip, the most probable first slip is somewhere along a line of maximum shear stress.

The first slip is followed by a redistribution of stress in its neighbourhood, thus throwing higher stresses on adjacent parts, with consequent slip. And so the slips multiply, each contributing to the total deformation observed. When at any place slipping has reached the stage where re-meshing is impossible, fracture takes place. The parts of the broken test-piece still retain their elastic property, although the space lattices, owing to innumerable slips, are damaged.

Interpretation of the Records.

Two experimental facts appear to identify elastic property with the space lattice. The first is the constancy of the elastic moduli, and the second is the behaviour of the single-crystal test-piece in which the influence of the block boundaries is eliminated. For example, although the strength and ductility of steel vary widely with the carbon-content, Young's modulus varies little with carbon-content, or even with mode of manufacture and heat treatment. The conclusion is that elastic property depends upon something much more fundamental in the metal than block size; and the Author suggests that it depends mainly upon the pattern and size of the space lattice. This view is supported by the study of Professor Carpenter's single-crystal test-pieces. Tests show elastic property and slip planes, so that the identification of elastic property with the space lattice is here direct and complete.

The following interpretation of the records with the aid of this conception of inner structure of metal indicates possible solutions of some of the problems presented by them.

Elastic Response to Reduction of Load.—This property of metal may be regarded as the effort of the space lattice to return to its original pattern with reduction of load. During slip, the lattice is distorted elastically to the extent corresponding to the slipping load, so that when the load is reduced slipping stops, and the lattice tends to recover its primitive pattern. The recovery is not truly elastic, because slipping has produced innumerable slip planes within the material, work is done by relative motion between them, and the lattice may be considered as damaged at these places of relative slip. When the load is entirely removed, the space lattices recover their primitive pattern, and the change in form—the permanent set—is a measure of the slip which has taken place.

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The Loop.—The loop recording the process of removing and re-applying the load may be regarded as a measure of the damage done to the space lattice by slip. Consider Fig. 28, Plate 2. The slope of the elastic line is the rate at which the lattice distorts with increasing load. At about the point p slip starts and gradually extends through the crystal blocks of the inner structure, with little or no increase in the elastic distortion of the lattices. The distortion corresponding to, say, q is maintained as slip proceeds. At A the load is reduced. Slipping stops, and the lattices shrink towards the primitive pattern, as shown by the elastic line A B. The space lattice has recovered its primitive pattern at B, and the slipping parts have re-meshed so that reapplication of load produces the slightly curved elastic line B to A. The area enclosed by the loop is a measure of the damage done to the space lattice by the extension to A. This area gradually increases to a maximum as stretch proceeds, as can be seen from Fig. 29, Plate 3. Maximum area of the loop probably corresponds to the condition where the maximum number of slip-planes have been produced within the test-piece.

The damaged space lattice possesses the power of mending itself when the metal is iron or soft steel. Recovery is accelerated by heating. With increasing carbon-content, or with the addition of alloys to the metal, this power of self repair becomes feebler and may disappear altogether. The looped diagram provides a simple method of testing this valuable property of steel. The method is as follows :—Take a record, say like Fig. 10, Plate 1, and then take the test-piece out of the machine and let it rest for a definite period. Then test again and draw a few more loops. Comparison of the areas shows the extent of the possession of the self-healing power. The Author has made this comparison when the rest period has been a year. High-carbon steels show no recovery even in this time. Irons and soft steels show recovery in a few hours.

Low Initial Resistance to Reversed Slip.—The push-pull and the torque-twist records establish a property of metal that is difficult to explain, namely, that when the load is reversed so that the direction of slip is reversed, the resistance to reversed slip is at first small, but gradually increases as slip proceeds. Referring to Fig. 32, Plate 3, at A the load is reduced to zero, and the response is the elastic line A B. At B the load is reversed to compression. The space lattice has been assumed to recover its primitive pattern at B. Now the reversal of the load distorts the lattice in the opposite direction preparatory to producing a reversed slip : but the resistance encountered along the slip plane is so low that a small elastic distortion is sufficient to produce the reversed slip. As slip

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proceeds, the resistance increases, so that the elastic distortion of the space lattice increases. The result is a curve which approaches to, and finally attains, the original value of the slipping resistance. This phenomenon is repeated every time the load is reversed. The conclusion that can be drawn from this is that reversed slip takes place along the planes established by the original first slip. It would appear from these results that when the load is removed, even after a moderate slip, the re-meshing of the space lattices along the slip-planes is imperfect, and that this imperfect meshing declares itself by the low resistance at reversal of load. Lapse of time in some material, and heat treatment in most materials, restore perfect meshing, and then the response to reversal of load is elastic up to the normal yield value, and even beyond it.

Conclusions.

The records emphasize the importance to the engineer of a knowledge of the resistance of a metal to slip. With the plastic contour of a metal before him, he can assign real factors of safety with certainty. The proportions of the parts of a machine or of a structure are determined with the aid of factors of safety which are intended to secure that the working stresses in the parts shall never exceed the elastic limit of the material. It is tacitly assumed that this condition is secured by using factors of safety based upon the ultimate strength of the material. The records show that, although the factor of safety used may be large enough to ensure that the working stress is well below the elastic limit, yet how far below is undetermined. This vagueness disappears if the factor is based on the slip resistance recorded in the plastic contour. The elastic limit is recorded on the diagram, provided the diagram is taken from a test-piece which has not previously been overstrained. For the steels in common use the elastic limit is 80 to 90 per cent. of the value of the slipping load.

Again, the plastic contour gives valuable information about the state of the metal tested. It is found that sometimes the plastic contour is not symmetrical about the origin when it is described between equal positive and negative limits of strain. The inference is, with reservations subject to future experiments, that the test-piece is in a state of internal stress when the test is applied. A single test on such a piece to determine the limit of elasticity would therefore give a false result, unless corrected by an experiment to find the limit with the opposite kind of stress. The true value would be the mean value found from an elastic

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experiment in tension and an elastic experiment in compression. The two experiments are unnecessary if the plastic contour is symmetrical about the origin. The test process to be applied would therefore be the application of equal positive and negative strains; and, if the resulting plastic contour were symmetrical about the origin, the elastic limit recorded would be the true one.

Again, the study of the plastic contour throws some light on what are regarded as mysterious failures. Frequently, in the processes of manufacture, parts of machines or structures are locally or completely strained beyond the limit of elasticity, so that the metal is left in an overstrained condition. In this condition the plastic contours show that the metal is in the worst possible state to resist the onset of a stress of opposite kind to that last applied. Inspection of any of the push-pull diagrams shows that a metal which has been even slightly overstrained in tension has been temporarily deprived of its elastic power to resist compression, and vice versa. The same applies to torque-twist records. This peculiarity of metal is specially illustrated by Figs. 31 and 32, Plate 3, which show the results of successive tests applied to a mild-steel test-piece.

Fig. 31, the first test, is the record of a stretch to 0·015 inch followed by a compression of 0·015 inch, then by a stretch to the primitive gauge-length of 2 inches, and finally by the removal of load. Fig. 32 is the record of exactly the same process applied to the same test-piece immediately after the test of Fig. 31. Although the plastic contours are practically the same in the two records, there is a remarkable difference in the lines starting from the origin. Fig. 31 shows a line curving away rapidly, in which there is no sort of proportionality between stress and strain: the yielding is plastic. But Fig. 32 shows a line from the origin with the characteristics of an elastic line. The stress is practically proportional to the strain up to the limit of elasticity. The explanation why these lines from the origin are so different, when the two diagrams are from the same test-piece and were described by the same process, must be sought in the previous mechanical history of the metal. The last load applied to the test-piece before Fig. 13 was taken was a compression, and this, being sufficient to cause slip, has crippled the elastic resistance in tension: but the last load applied to the test-piece before Fig. 32 was taken was a tension, which, being sufficient to cause slip, has crippled the elastic resistance in compression but not in tension, as the diagram shows.

The effect of this phenomenon may operate as follows. Suppose

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that a member which during manufacture has been overstrained in compression is built into its place in a structure. The first load it may be called upon to withstand is, say, a tension. It would stretch abnormally under this load and would therefore throw more than the designed stress on surrounding members. The result of the overstrain is therefore that at a crisis this piece does not contribute to the total resistance the proportion for which it was designed. Circumstance may easily be imagined where such a state of things would cause failure, and this failure would be quite inexplicable without a knowledge of this peculiar property of metal.

The restoration of equal elastic property in tension and compression can be conferred on overstrained metal only by heat treatment. In annealing there is safety of a peculiar kind. The subject of heat treatment is, however, no part of this Paper. It is a wide subject, full of industrial importance, and the study of it is always worth while, although manufacturers have already accumulated valuable data.

When a machine part possesses equal positive and negative elastic properties, alternating stress may be applied to it indefinitely, providing the maximum value of the stress never produces a slip in the space lattice. But a machine part overstrained by the necessities of manufacture and not afterwards heat treated may be called upon to resist a stress with its corresponding elastic resistance crippled.

The Author would suggest, therefore, that the combined study of elastic and plastic property is of value to practising engineers, and, further, that working stresses should be based upon a full knowledge of these properties as revealed in the plastic contour of a material.

A tensile looped diagram provides data for determining the value of E, and such information about the inner structure as may be inferred from the loop area and its rate of increase. A plastic-contour diagram, either of the push-pull or torque-twist variety, provides the data for fixing the slip resistance and therefore the working-loads and factors of safety. The simplest diagram of this type for practical purposes is the torque-twist; but much depends upon circumstances. Hydraulically-operated machines are more convenient for some kinds of test than is the hand-operated variety.

A series of torque-twist processes applied to test-pieces of 0·8-per-cent. carbon steel is shown in Figs. 33, 34, and 35, Plate 3. Fig. 33 is a single $\pm$ 4 degrees plastic contour; Fig. 34 is taken between a series of widening limits; and Fig. 35 is a record of

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looping between limits of ± 4 degrees. The lenticular areas are all alike and are symmetrical about the origin.

Figs. 36, 37, and 38, Plate 3, are a corresponding set from test-pieces of 0·22-per-cent. carbon steel.

All these records exhibit the characteristic plastic contours peculiar to the metals, which reveal to the engineer at a glance qualities of metals of profound importance in all kinds of constructional work.

The Paper is accompanied by forty-five photographs and four diagrams, from some of which Plates 1, 2, and 3, and the Figures in the text have been prepared.

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Discussion.

The President. The PRESIDENT, in moving the vote of thanks to the Author, remarked that it was the practice to invite distinguished members and visitors to take part in the discussions at the meetings, and for the Chairman to call upon individual members or visitors to speak, but the Council hoped that fact would not deter anyone not so invited or called upon from taking part in a discussion. The Council did not want there to be any feeling that because certain gentlemen were asked to discuss Papers, the discussions were not open to all.

The Author. The AUTHOR exhibited and explained a number of lantern-slides and wall-diagrams illustrating the history of the researches leading up to his Paper.

Dr. Unwin. Dr. W. C. UNWIN, Past-President, remarked that the Author had designed a set of testing-machines using an elastic weigh-bar to give the load indication, optical magnification, and photographic records. These exceedingly interesting devices—which must have involved a great deal of ingenuity in their designing—had accomplished one thing which, he thought, had not been accomplished before, that was, that the stress could be transferred from tension to compression, or vice versa, without disturbing the test-piece. Certainly the load-strain diagrams which the Author had exhibited were exceedingly beautiful: he did not think that such accurate diagrams had been taken before. If he looked to the conclusions drawn from the diagrams, he did not find very much that was novel, except on one point. The Author alleged—quite rightly—that the plastic deformations were due to slip at the cleavage planes of the crystals. That, he thought, had been known ever since Sir Alfred Ewing's research. But the Author further suggested that the elastic deformations were due to a deformation of the space lattices in which the molecules were placed in the crystal. That was quite new, and was an exceedingly interesting and, he thought, fertile suggestion. Engineers would be very glad to know that the elastic deformations and the plastic deformations were due to quite different actions. If the Author was right, the plastic deformations were mass actions and the elastic deformations were molecular actions. He recalled that Professor J. Bauschinger, of Munich, 35 or 40 years ago, obtained some results which threw light on the question of the interpretation of the pull-push diagrams. Professor Bauschinger,

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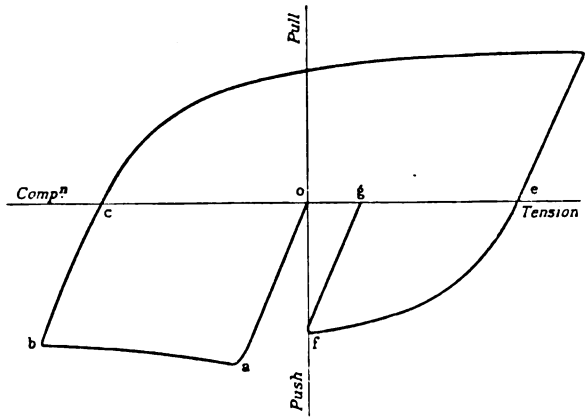
Dr. Unwin. who was probably the most accurate observer of the mechanical properties of materials who ever lived, found that the limits of proportionality of a specimen could be altered through a wide range by straining action: new elastic limits could be set up, but those new limits were rather unstable, and personally he (Dr. Unwin) would like to call them temporary limits. Next, Professor Bauschinger found that, if the elastic limit was raised in tension, it was lowered in compression; and, further, that if the bar were overstrained in tension, if loaded at once in compression, the elastic limit in compression was reduced to zero; and if it were overstrained in compression, the elastic limit in tension was reduced to zero. Fig. 16, Plate 1, illustrated those phenomena when compression preceded tension, and Fig. 32, Plate 3, when tension preceded compression.

Fig. 39.



Compression being applied, an elastic line o a was obtained (*Fig. 39*), the end a of the straight line being the initial elastic limit. Continuing, a plastic line a b was obtained, and a new temporary elastic limit in compression was established at b. This was shown because, when the load was removed, an elastic line b c was obtained. The line was slightly curved, and he attributed that to a time lag in the restoration of elasticity. Now tension was applied, and an entirely plastic line c d was obtained. No part was straight, so that there was no elastic limit at c. A new temporary elastic limit in tension was established at d, which was shown, when the load was removed, by an elastic line d e. At e compression was applied, and a plastic line e f was obtained. At e the elastic limit in compression was reduced to zero. A new temporary elastic limit occurred at f in compression, and, reducing the load, an elastic line

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f g was obtained. The final result of the cycle was a permanent set o g. That demonstration provided a very interesting confirmation of Professor Bauschinger's work. He thought that an explanation of the disappearance of the elastic limit after overstrain was given in the Paper. In overstraining the bar in compression there was a sliding at the cleavage planes of the crystals, say to the right. If immediately after overstraining, tension was applied, sliding occurred to the left, and there had not been time for the fissures on the cleavage planes to heal. They therefore yielded to the tension, however small it was, and the elastic limit was gone. On p. 39 the Author predicted dire results from using a bar which had been overstrained in the process of manufacture. It had to be remembered, however, that there was no evidence that overstrain in tension injured the bar for tensile stresses, and that overstrain in compression injured it for compressive stresses. There was a third case—when the bar was subjected to alternating stresses, like the piston-rod of an engine, and for that case it was known that the initial elastic limits in tension and compression were too high: the range of stress which the bar would stand under alternating stress was not so large as the range between the initial elastic limits. Therefore they were artificial limits, set up by the stresses which the bar underwent in manufacture. He did not think that contradicted Professor Bauschinger, for the reason that in manufacture the bar was subjected to stresses which were simultaneously compressive and tensile, because the bar had not freedom to expand or contract laterally, so that both limits might be raised. He congratulated the Author on his exceedingly interesting and valuable Paper. Dr. Unwin.

Sir ROBERT HADFIELD observed that no one had done more than the Author in the examination of the mechanical properties of steel, and the special apparatus described that evening showed great ingenuity in its design and construction. The Author rightly attached great importance to a thorough knowledge of the resistance of a metal to slip, and had found that for the steels in common use the elastic limit was approximately 80 to 90 per cent. of the value of the slipping load. The study of the behaviour of slip had been undertaken by numerous observers, and much information had been added to the knowledge on the subject. It was rightly claimed that by the Author's methods light could be thrown on many mysterious happenings; with that Sir Robert quite agreed, but he pointed out that it would be of further help if details could be added of the composition of the various steels tested. The Author had just shown on the screen a record of a test on a certain steel which was not satisfactory during the war, and which had had to be replaced Sir Robert Hadfield.

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Sir Robert Hadfield.

with a better quality. Naturally, metallurgists wanted to know the compositions of those two steels, since differences in their composition might explain the difference in physical behaviour. He did not know whether in that matter the Author was referring to steel used for munitions, but he could say, having had access to many private documents on the subject, that he was not aware that during the war any British steel failed after being made up into the final form of projectiles; while it was well known that of the steel supplied by certain other countries very considerable quantities had been found defective and could not be manufactured in the form of finished projectiles. He suggested that a number of specimens of steel taken from materials possessing known qualities (and of which the full chemical composition was known), of both good and bad types, should be examined by the Author's methods. He himself had collected a number of specimens of both types, and of which he had full records in regard to the actual behaviour of the materials in service. He would willingly place these at the service of the Author if he would carry out tests upon some of them. On the matter of the characteristics of the overstrained steel mentioned by the Author, he would like to refer to specimens he had tested some years ago[1]. One was of carbon steel and the other of manganese steel, the mechanical properties of which were very different. It was found that manganese steel, after being elongated 15 or 20 per cent. and still remaining unbroken, notwithstanding this straining of the material, had its qualities entirely restored by heat treatment and quenching. He had submitted one tensile test-bar to an extraordinary test, drawing it out 190 per cent. of its total effective length, with an elongation of about 30 per cent. at each separate application of the load. After each stretching, by heat-treating the material, its toughness appeared to be entirely restored. It seemed to him that this had a very important bearing on the particular point mentioned in the Paper with regard to the "slip" quality of iron and steel. He had also treated a bar of carbon steel in the same manner, and had found that such a test-bar could be drawn out about 120 per cent. of its length before breaking, that was, with 25 to 30 per cent. elongation at each application of stress. He thought, therefore, that the Author had used a happy phrase in speaking of the "self-healing" qualities of steel, which were fully proved in these two instances. By heat treatment he had been able finally to restore the original quality of the test-bar, that was to ay, the slipping could be healed by heat treatment. In that respect,

[1] Minutes of Proceedings Inst. C.E., vol. xciii (1888), pp. 30 and 34.

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therefore, he thought the Author had done excellent service in calling attention to the fact that after overstrain—not only to the minute degrees mentioned by the Author, but on the very considerable scale referred to above—it was quite possible to restore the quality of the material by proper heat treatment. He would not go into the important subject of heat treatment on that occasion; but the foregoing figures showed how desirable it was to submit steel which had been in any way strained to appropriate treatment of that nature. Sir Robert Hadfield.

Professor H. C. H. CARPENTER expressed his appreciation of the fact that the Author discussed his experimental results in terms of what was now regarded as the ultimate metal crystal unit, namely, the space lattice. Until recently the unit had been the crystal grain itself—obviously a very coarse structure, sometimes homogeneous and sometimes not; but according to modern conceptions the ultimate unit was the space lattice which, if the theoretical foundations were accepted, could be actually measured; and the property of steel and of all metals would ultimately be predicable in terms of the properties of the lattice. That was why it seemed to him so important that the Author, in the second part of his Paper, had attempted, admittedly in a speculative fashion, to account for the results which he had obtained. He was pleased to notice that Dr. Unwin accepted the Author's new point, namely, that the elastic distortion corresponded to the distortion of the lattice. If that was so, it was extraordinarily important. Professor Carpenter.

Professor E. G. COKER congratulated the Author on his Paper, and particularly on the beautiful diagrams he had shown, wherein he had repeated the cycle of stress very many times and had obtained practical coincidence. Only those who had tried to obtain such results would realize the mechanical difficulties of the problem. He did not think any other investigator had ever succeeded in getting so excellent a result. About 25 years ago Sir Alfred Ewing and Dr. James Muir had shown that the loops which were produced when the material was subjected to stress beyond the elastic limit could be at once reduced in area by immersing the material in hot water, and the plastic effect practically disappeared if the overstress was not great. It also disappeared naturally in the course of a few days. There was another important effect which had a bearing on engineering structures, particularly bridges, namely, the effect of cold on the "hysteresis" loop. He had had the opportunity about 25 years ago, when living in a very cold climate, of subjecting three test-bars of mild steel to overstress, and afterwards putting them out in the open air where for 2 months the temperature never exceeded —10°F. Professor Coker.

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Professor Coker. He found throughout that time that the loops, when produced under the same conditions of stress, were exactly like those produced before the bar was first taken out of the machine. It was necessary, of course, not to remove the ice which surrounded the bar, as a few minutes' exposure to ordinary room temperature made a difference. After such exposure to cold, a rise of temperature caused those loops to disappear. When the Author began to use collars for locating the gauge-points, it was a matter of interest to know their effect on stress-distribution. The photo-elastic record which had just been projected on the screen showed that there was some slight redistribution of stress—a rise of stress—in the neighbourhood of the collar. The flat celluloid specimen exhibited showed that there was a 25-per-cent. increase of stress for a very minute part of the contour at the collar, and that it was symmetrical on the two sides. Professor Coker did not know the amount of the increase in a round bar, but he thought the Author was entitled to say that the collar remained perpendicular to the axis of the specimen. An interesting point about the collar was that, no matter what the radius of the fillet might be, so far as he had made experiments on the matter, the additional stress was always nearly the same in every way. It was very different if specimens were undercut, and he had proved that undercutting caused a very considerable change in the stress—much more than was in general produced by collars. It was also interesting to note that the change in the stress at that particular section of the specimen was comparatively slight, except just at the contour. It was, in fact, an example of a numerous class of cases where additional material caused an increase, rather than a diminution, of stress.

Dr. Rosenhain. Dr. WALTER ROSENHAIN expressed his admiration for the ingenuity and beauty of the instrument which the Author described in its various forms, and for the interest of the diagrams which he had produced with that instrument. They were perfectly constructed, and carried investigators somewhat farther than the older work, to some of which Professor Coker had referred—the work of Dr. Muir done at Cambridge about 1899, and the work of Professor L. Bairstow on the behaviour of steel under cyclic reversals of stress, in connection with which he had obtained diagrams, not autographic, but broadly showing very much the same results as the Author's. Nevertheless, the possibility of obtaining such diagrams quickly and autographically was a valuable addition to the present means of investigating metals, and the Paper contained, he would not say the first fruits, but the early fruits of that investigation. Upon that the Author based an attempt at interpretation, and he thought, like Professor

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Carpenter, that he was to be congratulated upon that attempt, although Dr. Rosenhain did not feel he could accept the interpretation as correct in all respects. The Author was treading on safe ground when he said that the plastic yielding of metal occurred by slip in the crystals; but he went farther than that, and tried to imagine the mechanism of what occurred on the actual surface of slip. He did not do it in great detail, but he spoke of minute crystals having been "rubbed into individuality" at that surface. The Author was probably correct in that; but Dr. Rosenhain would emphasize that, if motion were forcibly caused on one or other of the geometrical planes of a carefully-arranged space lattice on which the atoms were held by mutually counterpoised forces in delicate equilibrium, it was not conceivable, in his opinion, that such motion could occur without profound disturbance in the immediate vicinity of that slip. The exact nature of the result of that disturbance was not known. The late Sir George Beilby, Assoc. Inst. C.E., had taken the view, and Dr. Rosenhain had supported it as far as he had been able by experimental evidence, that there was upon those surfaces a layer of material in which the space lattice was more or less completely destroyed, in which the atoms were pulled out of the proper positions and left in a more or less haphazard arrangement. How far haphazard was not known. It might be that there were merely a large number of fragments of lattice held together in some irregular fashion, but there must be considerable irregularity. If there was irregularity in the lattice near a slip plane, that irregularity must spread to a considerable distance, causing a distortion of the lattice; because any atom lying on the boundary of a lattice, whether that boundary had a free surface or was the boundary of a disturbed region where the atoms were not quite regular, must be pulled into a different position from that which it would normally occupy in an undisturbed lattice. It was known that such distortion of the lattice occurred when a stranger atom was introduced into it; and, just as it was known that such distortion caused hardening, so, he thought, it must be concluded that the disturbance of the lattice due to the local disruption on the surface of slip would also cause a distortion of the lattice, and consequent hardening. The distortion was not large, but a very slight distortion brought about considerable hardening. A distortion which, on the average, was only 1 or 2 per cent. was enough to make the difference between pure copper and a hard aluminium bronze. Consequently the investigator need not look for large distortions, but for distortions which only slightly affected the X-ray picture produced when the

Dr. Rosenhain.

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Dr. Rosenhain. aggregate was examined by modern methods. One point of considerable interest arose from the Author's suggestion, as the result of his experiments, that the resistance to reverse slip was less than the resistance to slip in the undisturbed material. That, he thought, was perfectly true, and some evidence confirmatory of that had recently been obtained by Miss J. McMinn and himself[1] when they examined iron under impact after it had been previously plastically strained. In that particular case the plastic strain was compression, and the impact following it was also compression, so that it was not a case of reversal. When a small piece of iron, which had previously had one surface polished, was strained by a blow, Neumann lines, not slip bands, were developed upon it. When it was strained slowly under compression in the same way, slip bands were developed upon it and no Neumann lines. When it was first strained plastically, even to a small extent just sufficient to develop slip bands, and then strained further in compression by impact, Neumann lines were not produced. New slip bands did not appear; there was merely an intensification of the existing slip bands, showing that the resistance to further deformation by slip, in the same direction and on the same plane, under impact was less than the resistance of the rest of the lattice. He was inclined to be cautious about explaining that fact, but none the less he would suggest the probability that on those surfaces there was a condition of material which allowed easy motion under certain conditions, but that after a time, when the material had recovered its elasticity, as the Author showed, easy motion was no longer possible. But he did not think it could be claimed—and that was where he differed from the Author—that the space lattice had the power of mending itself at ordinary temperature. There were many reasons for thinking that such was not the case. Cold-worked metal retained its distortion and its altered space lattices—its irregular arrangement—for years at the ordinary temperature, and it required a considerable rise in temperature before that re-arrangement of space lattices could really occur. All, he thought, that happened—at any rate he suggested it as a tentative explanation—was that the disturbed atoms were able in the course of time, and still better when their temperature was raised a little, to attach themselves in an irregular fashion rather more securely to their neighbours than they could do at the moment of disturbance. The absence of elasticity immediately after overstrain was a direct consequence of that disturbed condition of the material on the slip surfaces. It might be that, by the further development of methods of X-ray analysis, the possibility

[1] Proc. Roy. Soc., Series A, vol. cviii (1925), p. 231.

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would come about of examining, not as was done now, the average lattice structure over thousands or even millions of atoms, but in detail, step by step, the configuration of lattices. When that could be done, investigators would be able to give explanations in detail. Meanwhile the suggestions contained in the Paper were extremely interesting. He would like, however, to give a word of caution about them. The strains which the Author discussed were large ; that was to say, the plastic deformations were of the order of several per cent. The behaviour of materials under very much smaller strains, the kind of strain which led to failure under many millions of repetitions in alternating-stress testing for instance, might be, and he thought definitely were, different. Evidence had been given recently that repeated reversals of strain sufficient to produce slip in the first instance did not necessarily lead to fracture. It was not true to say that a reversal of a strain which produced slip must lead to fracture. What was more, when a stress of that kind, or even a stress just sufficient to produce ultimate fracture by reversal, was repeated, the number of slip bands increased largely, suggesting that under those conditions the old slip bands were not surfaces of easy reversal of slip. Dr. Rosenhain.

The AUTHOR, in reply, expressed his appreciation of the comments which Dr. Unwin had kindly made on the Paper. He had seized upon one point which the Author regarded as fundamental, namely, that the elastic property resided in the space lattice and was a different phenomenon from slip. As Dr. Unwin had very aptly said, "plastic deformations were mass actions, and the elastic deformations were molecular actions." The Author had always felt that the elastic property was something much more fundamental than slip, for, no matter how a piece of material might be treated, its modulus of elasticity was changed only slightly. Heat treatment produced wide changes in the physical properties of a metal, but almost negligible changes in the elastic property. Dr. Unwin's interpretation of Professor Bauschinger's work in terms of the results and suggestions put forward by the Author was very interesting. The inter-related properties of slip in compression and tension and the variation of the elastic limit could now all be shown on a photographic plate which could be taken with the Author's instrument in a few minutes, as contrasted with the long and laborious methods necessary to enable Professor Bauschinger to formulate his theory. It was surprising, however, how accurate were Professor Bauschinger's conclusions from the incomplete data which he had at his disposal. He desired to express his indebtedness to the work of Professor Carpenter and Miss Elam. It was common knowledge that they The Author.

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The Author. had succeeded in producing single crystals of aluminium large enough to be tested in a testing-machine. It was owing to the study of the behaviour of single-crystal test-pieces which he was testing for Professor Carpenter that the Author had been led to the conclusion that the elastic property of a metal could be identified with the space lattice and that the amorphous cement—or whatever it was—which held the crystal blocks together could be eliminated from the problem. The X-ray apparatus in use in Professor Carpenter's laboratory would help to extend still further knowledge of the properties of the space lattice. Sir Robert Hadfield had suggested that it would be an advantage to have the compositions of the various steels referred to in the Paper. The Author would refer him to p. 24, where analyses were given for the steels used for Figs. 5, 6, and 7. He was sorry he had not the complete analyses of all the steels from which the records were taken. His main object was not so much to correlate chemical composition with strength as to show how quickly the strength and elastic properties of a metal might be determined without any reference to its chemical composition. Among the diagrams which he had shown on the screen were the records of tests on two pieces of steel, which were supposed to be the same in quality. The diagrams, however, showed that one piece of steel was good; in the other piece the elastic property of the steel had disappeared. Although a metallurgist could draw conclusions from the analysis of the steel, he would still be ignorant of the amount of heat treatment and soaking which it had received during manufacture, so that any inference about its elastic properties could only be a surmise. But the elastic load-extension diagram revealed at once the difference in the elastic property of the steel. The diagrams told the engineer exactly what he wanted to know. With reference to Sir Robert Hadfield's remarks as to the connection between the chemical analyses and the mechanical properties of the steels, he would mention that he had reviewed[1] various metals used ordinarily in construction, and had shown for each metal the load-extension record, together with a microphotograph of the structure, side by side, the chemical analysis being added to aid in the correlation of strength, structure, and composition. Of course, if Sir Robert had any special piece of steel, the history of which was known, the Author would be very glad to see what sort of diagram could be obtained from it. Sir Robert had touched lightly on the question of heat treatment. This was a matter of profound importance. By the proper heat treatment of overstrained steels their elastic property

[1] "Strength and Structure of Steel and other Metals." London, 1923.

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The Author.

could be restored and their elastic limits raised, but, as mentioned, the Author had reserved the subject for further consideration because it was too large to be included within the limits of this Paper. Sir Robert Hadfield's remarks were specially interesting in respect of manganese steel. The Author had some load-extension diagrams of certain samples of this steel which Sir Robert sent to him some time ago, and they showed that it possessed remarkable properties. He was glad that the speculations he had based on the work of Professor Carpenter did not appear to him to be unjustifiable. In referring to the comments of Dr. Unwin he had already shown the great importance to the engineer of Professor Carpenter's researches in both practical and scientific knowledge of the strength of materials, and also the influence of those researches on some of the conclusions given in the Paper. Professor Coker's interesting experiments on the effect of cold on the "hysteresis" loop furnished further evidence of the fact that the raising of temperature had the effect of causing the slip planes produced by over-extension to remesh. The slide made for the Author by Professor Coker and exhibited at the meeting showed the effect of collars on test-pieces, and the slight modification of stress at the root produced thereby. Professor Coker's valuable remarks regarding the radius of the fillet showed that no great accuracy was necessary in forming the collars. The surface of the collar on which the fingers of the extensometer rested must be, of course, at right-angles to the axis of the test-piece and must be finished smoothly. Dr. Rosenhain's remarks were of much interest, and the Author was very glad that the researches of Dr. Rosenhain and Miss McMinn had confirmed the results he had obtained from his push-pull and torque-twist tests, namely, that resistance to reversed slip was so much less than the resistance to slip in the undisturbed state. Dr. Rosenhain had stated that "he did not think it could be claimed—and that was where he differed from the Author—that the space lattice had the power of mending itself at ordinary temperature." The Author would point out that this statement was evidently due to a misapprehension. He did not for a moment mean that the space lattice had in general the power of mending itself at ordinary temperatures. On the contrary, he had published the results of some definite experiments to show that both carbon steels and alloy steels did no mending at ordinary temperatures, but required heat treatment to bring about re-meshing[1]. At the same time, it was well known that

[1] "Researches on the Elastic Properties and the Plastic Extension of Metals." Phil. Trans. Roy. Soc., Series A, vol. ccxxi (1920), p. 117.

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The Author. soft irons and mild steels did mend themselves and recover their elasticity by mere lapse of time[1]. A comparatively slight change of temperature in a metal, due, for instance, to boiling in water, accelerated the process of mending[2]. In the Paper by the Author referred to, examples were given of two test-pieces; one of them recovered its elasticity at ordinary temperatures in a night, and the other, when tested after a year's interval, was found to give exactly the same "hysteresis" loop as when tested 12 months previously. With regard to Dr. Rosenhain's further remarks, when a load was applied to a test-piece and its removal and re-application formed a loop, even although the yield-point had not been passed, the true elastic limit had been passed, and unless the space lattice mended itself or was subjected to some heat treatment, the Author did not think it safe to say that it would be able to withstand reapplication and removal of the load indefinitely. It should always be remembered that a fatigue test was not so much a test as a process; and the results obtained by this process required careful interpretation before they could be accepted in relation to the use to which the metal was to be put. In conclusion, he would like to emphasize the fact that the recorders mentioned in the Paper were not designed primarily for purely scientific research but for the purpose of giving engineers quickly and accurately information about the elastic and plastic properties of the metals which they used.

Correspondence.

Mr. Batson. Mr. R. G. C. Batson remarked that the Author had expended considerable care and thought upon the design of his optical instruments and had given numerous interesting records in the various Papers which he had written on the subject. He was interested to find that the Author advocated the application of positive and negative strains in order to obtain a true estimate of the elastic properties of materials. Work in this direction had been carried out, in alternate tension and compression, by Professor L. Bairstow[3] and in alternate torsion by Mr. Batson[4]. By twisting the bar in one direction beyond its limit of proportionality (stress x) and leaving it in boiling water for 30 minutes, Mr. Batson had found that the limit of proportionality was raised to (or "fixed" at) the maximum stress x_1 to which the bar had been subjected. The bar

[1] Proc. Roy. Soc. vol. lviii (1895), p. 134.
[2] *Ibid.* vol. lxiv (1899), p. 337.
[3] Phil. Trans. Roy. Soc. Series A, vol. ccx (1909), p. 35.
[4] Proc. Inst. Mech. E., 1917, p. 182.

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was then twisted in the opposite direction, and the limit of pro- Mr. Batson.
portionality, y, was obtained in that direction. The process was repeated a large number of times, and in all cases the range $(x_1 + y)$ was found to be constant. That was to say, if the "fixed" limit of proportionality was high in one direction, the limit of proportionality, found when the twisting was reversed, was low. Further, basing his conclusions on the work done by Professor Bairstow in alternate tension and compression, it was assumed that the range $(x_1 + y)$ corresponded to the limiting range of stress in repeated alternating shear. This assumption had been verified subsequently by Mr. H. J. Gough,[1] by actual fatigue tests (in shear) on the materials used for the work cited above. The actual estimation of the limits of proportionality in the work carried out by Professor Bairstow and himself had necessitated the measurement of very small strains with a high degree of accuracy. This requirement was indeed essential in all work dealing with the proportional or elastic limits, and Mr. Batson was doubtful whether, owing to the fundamental characteristic of the optical instruments, to which the Author referred (namely, a spherical error due to the spot tracing a curve on the photographic plate), the optical elastic recorder could give results accurate enough for research purposes. The observations were, however, probably sufficiently near for commercial work.

Fig. 15, Plate 1, showed that the yield load was not quite the same in tension as in compression. On comparing that diagram with Figs. 12 and 13, Mr. Batson had been unable to decide which kind of strain the Author found to give the higher value, as the latter diagrams appeared to give identical values of the yield. Theoretically the yield should be higher in compression than in tension, and some careful extensometer tests by Mr. Batson had shown such to be the case. These tests were given in the following Table, and represented average values from three tests (in close agreement) in each case.

COMPRESSION.

Material.	Limit of Proportionality.	Yield Stress.	Modulus of Elasticity.
	Tons per Sq. In.	Tons per Sq. In.	Tons per Sq. In.
Forged Steel A	17·2	19·6	13,030
Forged Steel B	20·3	20·6	13,120
Pure Nickel Bar	6·6	8·6	12,940

[1] "Fatigue of Metals." London, 1924.

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Mr. Batson.

TENSION.

Material.	Limit of Proportionality.	Yield Stress.	Ultimate Stress.	Modulus of Elasticity.
	Tons per Sq. In.	Tons per Sq. In.	Tons per Sq. In.	Tons per Sq. In.
Forged Steel A	16·9	18·5	33·3	13,300
Forged Steel B	16·8	19·9	34·8	13,390
Pure Nickel Bar	5·6	8·4	32·4	13,210

Professor Cook.

Professor GILBERT COOK considered that it was remarkable, in view of the amount of research and study that had been devoted to the mechanical properties of mild steel, that a state of considerable uncertainty still existed regarding the process of failure under stress, and the laws which governed it. It was, perhaps, not difficult to visualize what took place at rupture. Whatever might be imagined to be the ultimate molecular arrangement, it was possible at all events to conceive of the molecules being torn apart by the application of sufficient force. Nor was it difficult to conceive of the process of elastic deformation. Any form of space lattice in which the molecules were held together by mutual attractions must deform in some way under the application of external forces, and it was readily conceivable that such deformations might be proportional to the applied forces up to a certain limit. The difficulty lay in trying to get some idea of what was happening between the elastic limit and the point of rupture, and particularly what happened at the yield-point. In this connection Figs. 8 and 10, Plate 1, were of special interest. Fig. 8 was of the familiar type obtained with any kind of autographic recorder, but it exhibited very well the feature which, among all materials, appeared to be possessed only by mild steel and wrought iron in the annealed state; that was, the sudden reduction in the stress immediately after the yield-point. In collaboration with Professor Andrew Robertson, Assoc. M. Inst. C.E., he had described[1] experiments which had been carried out in order to explore this region. A diagram very similar to Fig. 10 was obtained, but not by autographic means. It was found, however, that under certain conditions a much greater reduction occurred than was indicated in the Author's diagram, the amount of the reduction ranging chiefly between 24 and 30 per cent. of the yield stress, but being as high as 35 per cent. in two specimens

[1] Proc. Roy. Soc., Series A., vol. 88 (1913), p. 462.

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Professor Cook.

The material used was ordinary commercial mild steel, but it was carefully annealed, and special precautions were taken to secure a uniform distribution of stress during loading; also the extension was increased at a very slow rate. The rate at which the extension was increased appeared to affect the amount of the drop in stress, a fact which was important in considering any theory of failure. The results had been confirmed by a torsion test of the material. A torque-twist diagram almost identical with that shown in Fig. 7 was obtained. As the torque was increased, the angle of twist was proportional to it up to a point at which elastic breakdown occurred in the outer layers, and it was found then that the twisting continued without any further increase in the torque; in other words, the torque-twist curve became a horizontal line starting from the yield-point. If x denoted the ratio of the stress immediately after, to that immediately before, the yield-point T, T' the torques at, and immediately after, the yield-point, and θ and θ' the corresponding angles of twist, it could be shown that

$$T' = T\left(\frac{\theta}{\theta'}\right)^3 + \frac{4x}{3}T\left\{1 - \left(\frac{\theta}{\theta'}\right)^3\right\},$$

from which it would be seen that T' would be equal to T if x were equal to 0·75; that was, the twist would continue without increase of torque only if the drop in stress in the layers which had yielded was equal to, or greater than, 25 per cent. There was evidence, therefore, once the process had started, of a comparatively low resistance to sliding, which appeared to diminish when the rate of sliding was reduced. This fact lent support to the theory put forward by the late Sir George Beilby, whereby it was sought to explain the process of sliding by the occurrence of a change in the physical state of the material adjacent to the planes of sliding—a transformation from the crystalline state to an amorphous state. One of the properties of this amorphous material was assumed to be that of a highly viscous liquid while sliding was taking place, and of considerable hardness when the sliding stopped. In that connection Fig. 10, Plate 1, was extremely interesting. When the load was removed and reapplied, the extension continued at approximately the same stress as that which existed just prior to the removal of the load, although that was considerably less than the yield stress. It followed that the sliding must continue over the same crystal planes, and not over new ones, otherwise the original yield stress at least would be required to re-start the movement. It would appear, therefore, that the hardened amorphous state could be reconverted to the viscous state, and that in the period occupied in the removal and reapplication

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Professor Cook. of the load no appreciable recrystallization of the amorphous material could have taken place. It would be interesting to know whether the same effect would be produced if, while still in the region of the "drop," the load were removed, and then reapplied after a prolonged interval of rest, and possibly heat treatment at 100° C. Would the new yield-point have the primitive value or the reduced value? The answer to that question would undoubtedly throw additional light on the properties of the material adjacent to the planes of sliding. On the hypothesis of an amorphous material having the properties described above, there did not appear to be any serious difficulty to encounter in interpreting the Author's diagrams. The quasi-elastic response to reduction of load was what would be expected as a necessary consequence of the cessation of sliding and the production of the hardened state. In those parts of the material remote from the planes of sliding the distortion was purely elastic, and the removal of the load would be an elastic process modified to some slight extent by readjustments in the amorphous layers. The removal of the load would not, however, relieve the material of internal stresses. The process of sliding must produce considerable complication in the stress-distribution in the material, and the mere process of removing the load would be to generate compressive stresses in those regions which had sustained a comparatively low tensile stress when under tensile load, as, for example, in the amorphous material itself. A reversal to compressive load would increase such compressive stresses, and it was readily conceivable that a very small compressive load would bring such material again to the conversion point, and slipping would occur again over the same planes, but in the reversed direction; and a diagram like Fig. 16, Plate 1, would be obtained. The amorphous state was as yet little more than a hypothesis. But it was one that was consistent with a great deal of external evidence, which was very largely contributed to by the Author's results.

Mr. Doak. Mr. W. J. Doak observed that the Author's picture of the internal structure of iron and steel and of the difference between the fractures of single-crystal and multi-crystal test-pieces was very interesting, and it occurred to him that it might be worth while to investigate the effect upon the fracture if a test were carried out on soft iron in a strong magnetic field. The well-known effect which hardening had upon retention of magnetism seemed to suggest that the arrangement of space lattices might be responsible for both strength and magnetic properties, and that variation of one would affect the other. The weakness of magnetic force as compared with tensile strength indicated that the effect would be slight: still, with the Author's delicate apparatus some interesting results might be hoped

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for. Mr. Doak did not notice any reference to "yield-point" or to "limit of proportionality"—terms which had been considered to be of serious importance by the engineers of Sir William Arrol and Company in connection with their tender for Sydney Harbour bridge. In fact, although on p. 25 proportionality was stated to cease at 20·4 tons, there was exceedingly little visible bending in the straight line of Fig. 6, Plate 1, till the peak load was reached at 23·8 tons per square inch. In this connection it would have been interesting to stop the test at, say, 0·009 inch extension and reapply the load; probably then the diagram would have been quite straight, with a permanent set at no load, and the limit of proportionality would coincide with the yield-point. In regard to practical affairs, every bridge-engineer was familiar with cold bending of parts of bridges which had been damaged in transit: there could be no question that such parts were strained beyond the elastic limit, and it then became a matter for judgment whether they should be annealed before going into the bridge. Whether the part was subject to reversal or not, the Author's tests showed that resistance was weakened for load of an opposed kind; and, as a bent plate would have been strained in both tension and compression, there must be a lessened resistance to strain of both kinds. Fortunately, however, as soon as the slack, so to speak, was taken up, the plate became again elastic material, and the only bad effect would be a certain deformation of the girder. As already said, judgment must be used so as to steer a course between taking too much risk and incurring needless expense in annealing. Railway rails afforded a practical instance of the use of permanently-strained material to carry heavy loading. Where lines had sharp curvature the rails were universally pressed cold to the precise curvature, although on flat curves it was common practice merely to spring them without permanent bending. In addition to this, the concentrated pressure of wheels caused the rail-surface to flow, so that it was quite common to see the metal rolling down the sides in the form of a bead; yet those rails suffered heavy reversals of stress under every passing train. It was not as though the stresses were low, because extended investigations in America had proved that very high stresses occurred under ordinary traffic-conditions. These facts seemed to suggest that the factors of safety and allowances for impact and dynamic effect in the design of bridges (highway bridges in particular) were probably greater than was necessary. In his experience several important bridges were carrying loads much in excess of those for which they had been designed, and it would seem more logical to cut down the impact-allowance and to include in the factor of safety definite allowances for possible increased loading and reduction by corrosion. In that way consistent rules for condemning

Mr. Doak.

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Mr. Doak. old structures might be evolved. At present there was a wider gap than might be expected between the loads for which bridges were designed and those they could carry, taking into consideration the better knowledge of theory and extended experience now available.

Sir Alfred Ewing. Sir J. Alfred Ewing congratulated the Author warmly on the perfection to which he had brought the photographic stress-strain recorder and on the applications to which he had put it. He could speak with some knowledge of the difficulties that were met in obtaining a satisfactory record by giving two components of angular motion to a pencil of light, for it was a problem he had attacked years ago in designing his magnetic curve-tracer. In the Author's device much finer and better diagrams were obtained, which for precision and clearness left nothing to be desired. The Author had, in fact, placed in the hands of engineers a really valuable instrument of research. It was also an instrument which might, and should, greatly simplify practical testing. In cases where large forces had to be applied, as in tension tests of chain cables or compression tests of columns, there was no longer any need to resort to testing-machines provided with a lever, or system of levers, and weights for the purpose of measuring the applied force. That could now be better done—more conveniently and at least as accurately—by using a steel master-bar or bars in tension, and taking the master-bar's extension as the measure of the force applied. This allowed the design of the testing-machine to be immensely simplified. It meant substituting, as the means of measuring the hydraulically applied force, a spring in its simplest form—that of a rod in tension—for the complication of knife-edges and fulcrum planes, levers, and movable weights. Thanks to the Author's recorder the old-fashioned type of testing-machine for large stresses would give place to a handier, less costly, and equally effective appliance: indeed, a more effective appliance, for it would not only measure the stress with at least as great precision, but would record the strain suffered by the object under test, at every stage from start to finish. There was, of course, nothing new in this idea, but the point was that the Author's device made it thoroughly practicable. He commended it to the attention of all engineers who were concerned with large-scale testing. He was struck by the close resemblance of the Author's diagrams for cyclic processes of straining in steel and other metals to another class of diagrams with which he was very familiar, namely, hysteresis curves showing the variations of magnetism produced by cyclic variations of magnetizing force. The correspondence was remarkable, not only as regarded general form, but also in detail—as, for instance, in the progressive differences which were observed between one cycle and the next when a straining, or magnetizing, process was repeated.

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The curves of strain and stress were so exactly like hysteresis curves that any one accustomed to magnetic testing might, on glancing at the plates without reading the letterpress, mistake these for records of magnetic tests. Surely so close a resemblance implied some fundamental community in physical origin. It would seem that the interatomic forces which controlled the response to mechanical straining must act in an essentially similar manner to those which controlled the response of the magnetic elements in the atoms of an iron crystal to the turning influence exerted by an externally-applied magnetic field. Sir Alfred Ewing.

Mr. F. L. WATSON considered that the Paper, in continuation of the Author's previous publications, furnished a most valuable step in the solution of the urgent problem of a simple and rapid means of co-ordinating the results given by the various mechanical tests already in use, so as to arrive at the vital point in regard to the strength of material, namely, the maximum load which might be applied to the material without weakening its internal structure. Opinions on the well-known mechanical tests varied between the extreme views that simple tensile tests would give all the information required if properly interpreted, and that the shock-breaking test should supersede all others. The instruments devised and used by the Author in these researches had the advantage that the test might be carried out at any desired speed, whilst the complete record could be examined at leisure ; and the absence of friction and inertia in the apparatus ensured that speed of testing could introduce no error into the record. The actual handling of the apparatus was simple, and errors due to differences of touch did not arise : although special skill and care were required in the original construction and calibration of the instrument, its actual use involved no anxiety, but only ordinary care. A comparison between the diagrams obtained by the Author with these instruments, and those obtained with the stress-strain recorder of the late Mr. J. Hartley Wicksteed, M. Inst. C.E., which was widely used in single-lever testing-machines, showed the remarkable conformity of the forms obtained with the earlier recorder with those obtained by the far more accurate instruments of the Author. The much higher magnification of strain-scale in the Author's elastic instrument rendered it peculiarly suitable for the important research work disclosed by the Paper. The Author's instruments were now put to various uses in several very important research laboratories, in addition to his own, and Mr. Watson felt that it was only a question of time before the results of these researches would find a place in the standard specifications used by engineers. Mr. Watson

The AUTHOR, in reply, remarked that the valuable results obtained by Professor Bairstow (which Mr. Batson had mentioned) might be The Author.

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The Author. taken as showing that the elastic range must fall within the plastic contour of a metal. That, of course, was in accordance with the later information given by the diagrams in the Paper. He thought Mr. Batson had omitted to notice that the spherical error due to the spot tracing a curve on the photographic plate was not carried into the data derived from the records, because, as mentioned on p. 22, the diagram was read by means of a grid plate which was placed upon it, and which had been calibrated in situ, so that the spherical error was eliminated. Such an error would arise only if the records were measured with linear scales, but that was never done. The Author's experience led him to believe that the recorders, and especially the elastic type, had greater possibilities for research than Mr. Batson would appear to believe. The standard forms of laboratory extensometer suffered from two fundamental disadvantages: the loading was not continuous, and the observations were plotted by an observer who drew the best curve he could through the dots, so that personal error was introduced. To these might be added a third disadvantage, namely, the time occupied in obtaining even one curve. When the load was put on in steps, during the pause when the strain was being observed, creep began in the region of the limit of elasticity, and therefore the stretch was strictly a function of the time. Consequently, a step-by-step observation was very liable to give anomalous results when reduced to a curve. The Author was convinced that the recorders gave more accurate results for the purpose of most researches into the strength of materials than did other instruments now in use, and they possessed the further advantage of rapidity of action, which specially suited them not only for commercial work but also for scientific investigations involving a large number of tests. He felt sure that such a diagram as Fig. 26, Plate 2, could not be obtained with any other existing instrument. That diagram had been taken and the photographic plate developed in a few minutes. He did not think that the Table given by Mr. Batson, showing the difference between yield in tension and yield in compression, could be regarded as conclusive. That difference might be due to initial stress in the metal or to other causes. From the very large number of plates which the Author had taken he was led to believe that there was now practically no difference between the yield in tension and the yield in compression, especially in the higher carbon steels and the alloy steels.

Professor Cook had directed attention to an important property of iron and mild steel, namely, the sudden reduction of stress immediately after the yield-point had been passed. When the diagrams obtained with the Author's recorders were first shown, he had to meet the criticism that the indication of this reduction was due to instru-

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mental error; but Professor Cook's own experiments confirmed the accuracy of the records and showed that this drop was a property of the material. Professor Cook pointed out that the amount of reduction ranged from 24 to 30 per cent. and that he had obtained reductions as high as 35 per cent. The Author quite agreed; but the amount of reduction, of course, was not constant; it varied even in specimens cut from the same bar. In the hundreds of diagrams which he had obtained from iron and mild steel he had never found this property absent when the metal had been annealed, but it disappeared if the metal had been overstrained. His experiments showed that this was not a general property, but was peculiar to iron and mild steel, and that the sudden reduction of stress at yield became less as the carbon content was increased, until in the alloy steels it disappeared altogether. But he did not think that any general conclusion could be drawn without making a large number of experiments; he had found, for example, that the magnitude of the drop increased when the rate of loading was increased. In a record taken by him 6 years ago from a piece of Yorkshire iron, the drop, which was normally 10 per cent. when the specimen was broken in about 30 seconds, increased to 30 per cent. when it was broken so quickly that the yield was passed in approximately 2 seconds. He had always associated a large drop with acceleration in the rate of applying the load. That record, like all other records, had not been obtained in the ordinary lever testing-machine but in an hydraulic press, specially designed for use with the recorders. The torsion results and the deductions drawn therefrom were very interesting. He presumed that the formula would have no application to alloy steels; in fact, to be capable of general application it would have to have the composition of the steel brought into it.

The Author.

Concerning Mr. Doak's remarks on the yield-point and the limit of proportionality, the Author had not tried to define these points, or indeed any points at all. It would be very difficult to do so satisfactorily, because the same terms were used in different senses by different workers. He found that it was much better to present the curve showing the elastic properties and to leave nomenclature alone. A definition of the yield-point which would apply to iron and mild steel would not apply to alloy steels. So far as the elastic property was concerned it was more satisfactory to say that, if the load could be removed and reapplied without producing a loop in the diagram, then the limit of elasticity of the material had not been exceeded; and that, if a loop appeared as a result of this process, then the limit of elasticity had been passed. The change from the elastic to the non-elastic condition was usually so gradual and subtle that it was difficult to identify it. As Mr. Doak had stated with

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The Author. reference to Fig. 6, Plate 1, it was difficult to say where yield began, but it was quite certain that the limit of elasticity had been passed before the yield load. This could be shown by leaving a load less than 23·8 tons per square inch hanging on the test-piece; it would be found to creep with any load between 20 and 23 tons per square inch. Reference might be made here to the Author's Paper[1] on "Research on the Elastic Properties and the Plastic Extension of Metals."

Mr. Doak had referred to the cold bending of parts of bridges, which occurred in transit. The material used in making bridges was low-carbon steel which possessed the power of elastic recovery with mere lapse of time. Annealing was therefore unnecessary. But any parts made of high-carbon or alloy steel should be annealed, as those materials had not the power of elastic recovery with lapse of time. Reference might again be made to the Paper mentioned above[2]. The object of the extended research which the Author had undertaken was to correlate the mechanical tests now in use. He did not doubt that a little experience in the use of his recorders would convince anyone that supplementary tests were in most instances unnecessary.

He was particularly glad to read the remarks of Sir Alfred Ewing, and he would like to acknowledge the inspiration and encouragement which he received from Sir Alfred when he assisted him in the development of the Engineering School at Cambridge University.

As both Sir Alfred and Mr. Doak had pointed out, the resemblance between the stress-strain diagrams for cyclic processes of straining in steel and other metals and hysteresis curves showing the variations of magnetism produced by cyclic variations of magnetizing force were remarkable. The tracing of the fundamental community in physical origin of these curves did indeed open up a field of research which had still to be explored, but the Author had not yet been able to undertake this work.

He was gratified that his conviction concerning the possibilities of the recorders was shared by such an eminent investigator in this subject as Sir Alfred Ewing, whose statement that the Author had, in fact, placed in the hands of engineers a really valuable instrument of research showed that not only did the rapidity of action of the recorders suit them for commercial work, but they were equally useful for research.

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[1] Phil. Trans. Roy. Soc., Series A, vol. ccxxi (1920), p. 134, Fig. 24.

[2] *Loc. cit.*, p. 121, Fig. 4.

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1 December, 1925.

Sir WILLIAM HENRY ELLIS, G.B.E., D. Eng., President, in the Chair.

The Council reported that they had recently transferred to the class of

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CECIL JOSEPH PARKER, *M.C.*
JOHN PARR, B.Sc. (*New Zealand*).
JOHN WHITE SHAW, B.A., B.E. (*Royal*).
JOHN HENDERSON TAYLOR.

And had admitted as

Students.

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ROYDON FARRANT ALLEN.
HENRY STUART ANDREW.
JOHN BAKER.
BASIL HYDE BARKER.
HERMANN BARNETT, B.A. (*Cantab.*).
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JOHN WILLIAM WEITZ.
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RONALD MACKAY WOOD, B.Sc. (*Manchester*).
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NORMAN LEIGH WYATT.
WILLIAM FAIRLIE YOUNG.

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HASAN ZAKY, B.Sc. (*B'ham*).

The following Paper was submitted for discussion, and on the motion of the President the thanks of The Institution were accorded to the Author.

(*Paper No. 4541.*)

"Trafford Wharf Reconstruction at the Manchester Docks."

By HENRY ASHMAN REED, M. Inst. C.E.

WHEN the Ship Canal was made, all the docks at Manchester were constructed with mass-concrete walls, except the south side of the dock area from Mode Wheel Locks to Trafford Road swing bridge (Fig. 1, Plate 4). Along part of this length a timber wharf, known as Trafford wharf, was constructed on the canal slope in 1893, before the water was let into the dock area, and this wharf was extended also in timber after the Ship Canal was opened. The wharf, which had a total length of 2,362 feet, was constructed to several different designs.

It is not within the scope of this Paper to discuss the reasons for adopting the different designs on their respective merits, but only to consider their bearing on the design and construction of the wharf by which they have been replaced. The timber deck of the wharf was level with the ground at the rear, and was used continuously for the unloading of timber direct from ships by means of their own gear. There were no cranes on the wharf, which was, however, provided with roads and railways at the back to serve the timber stacking ground.

The whole of the pitch-pine timber in the wharf below water-level was found to be in excellent condition, and those piles which, being in the way of the new work, had to be drawn, were all used elsewhere. The timber between wind and water was in very bad condition, as was the timber where moisture could collect and remain. Timber above water which was in positions where it could dry quickly after rain was in fair condition, but not nearly so sound as that under water.

The Co-operative Wholesale Society, Ltd., have a large system of flour-mills a short distance from the back of Trafford wharf at

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its upstream (or east) end, and in 1912 a length of 510 feet of wharf was reconstructed, in reinforced concrete, jointly by the Canal Company and the Co-operative Wholesale Society, in order to afford facilities for the unloading of grain direct from ship to flour-mill by means of two travelling bucket elevators, each of 100 tons per hour capacity, discharging to bands in elevated galleries, by means of which the grain is delivered to the flour-mills at the rear of the wharf.

The remainder of the wharf—1,657 feet long—continued to be used for the unloading of timber. During its life the superstructure had been repaired from time to time, but in 1921 either very heavy repairs or reconstruction was found to be necessary. After careful consideration it was decided that this frontage to the canal in the dock area was too valuable to be used for timber discharge and storage only, and a comprehensive scheme was therefore designed to utilize the site to the best advantage for general cargo purposes.

At the west end of the wharf a berth for unloading whole grain cargoes was provided in 1898, with a tower housing a bucket elevator to unload grain from ships at a maximum rate of 350 tons per hour and discharge it to a grain-silo, with a capacity of 40,000 tons, situated about 110 yards away from the quay-side. The bucket elevator is supplemented by pneumatic plant with a capacity of 120 tons per hour—a useful adjunct for dealing with small parcels, for negotiating awkward hatches, and for cleaning up ships' holds.

The whole frontage to the docks to be considered in the scheme of reconstruction, including the grain-berth and the old timber wharf, was 2,100 feet. In designing the new wharf, it was considered desirable to provide facilities for the discharge of part grain cargoes to the existing grain-silo so that the grain might be discharged simultaneously with other cargo, as is already done at No. 9 dock, Manchester, where subways are provided in the dock-walls, with belts delivering grain to No. 2 silo, holding 40,000 tons, at the head of the dock. This system eliminates the delay occasioned by having to move vessels to a special berth for unloading grain after other cargo has been dealt with.

In order to accomplish this object it was necessary to design the wharf with a grain-subway practically along its whole length. It was also decided to scrap the existing tower with the bucket and pneumatic elevators for whole grain cargoes, and to extend the wharf and subway along that length of frontage as well. At the back of the wharf transit-sheds are to be built, with roads, railways, cranes, capstans, and all the necessary facilities for dealing with general cargo.

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The complete scheme consists of a wharf 2,028 feet long with two lines of railway, a range of four five-storey transit-sheds, each 450 feet long by 132 feet 6 inches wide, cranes back and front, and all the necessary cart-roads and railway-sidings for dealing with the traffic in and out.

A subway is provided along the front of the wharf for electric cables, water-mains, etc., with man-holes and plug-boxes for crane connections at the deck-level. A larger subway carries the grain-handling plant, and cross subways deliver to the existing grain-silo, which will have to be partly redesigned and equipped with additional elevators, weighing-machines, and distributing-belts to deal with the additional streams of grain entering the building.

Description of New Wharf.—It was further decided to reconstruct in the first instance the existing wharf, with the foundations for the two central transit-sheds, and afterwards to proceed with the superstructure of the two sheds, leaving the replacement of the existing grain-unloading berth with a wharf and the construction of the two remaining transit-sheds for future development.

The facts that the wharf was to be built in the wet, that a subway of large size was necessary, that the height of the quay above the water-level was restricted to about 9 feet 6 inches, and that the structure must be of ample strength to resist the blows of vessels, necessitated special treatment; and the design is, therefore, unusual in character. Reinforced concrete was adopted to meet these requirements, and as the height is restricted and it is difficult to construct shuttering and to mould reinforced-concrete members in situ close to the water, especially where there are continual fluctuations in the level, it was considered desirable to use pre-cast units for the length of the wharf structure on which the main subway was to be constructed. A short length of wharf at the east end, where the grain-subway was not required, was designed for ordinary concrete pile construction with reinforced-concrete framing and deck.

The length of wharf which included the grain-subway was founded on groups of piles surrounded by concrete cylinders filled with mass concrete (Fig. 2, Plate 4). The superstructure was formed of two rows of pre-cast concrete boxes 25 feet long, placed on the cylinders, and carrying the grain-subway between them. The back of the wharf is carried on the piled foundations which also carry the front portion of the transit-sheds, and the whole structure is tied together by the reinforced-concrete deck (Figs. 3, 4, and 5, Plate 4).

The depth of water in the Manchester Docks is 28 feet. The major portion of the slope under the existing timber wharf is 1½ to

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1, and is pitched with rubble stone of an average thickness of 1 foot 6 inches. The timber wharf was not brought out to the toe of the slope, the face line being about 5 feet behind it. It was necessary, however, to fix the line of the new wharf at the toe of the slope so as to give 28 feet of water; thus ships of 26 feet draught could use the wharf freely, and the basin fronting the wharf could be dredged up to the face line without fear of disturbing the existing slope of rubble pitching.

The two lines of cylinders are located 25 feet apart between centres longitudinally and 23 feet 9 inches transversely. The reinforced-concrete cylinders in the front row are $4\frac{1}{2}$ inches thick and 5 feet 9 inches internal diameter, founded on clay at canal-bottom; they are 28 feet 2 inches high, the top being 2 inches above ordinary water-level. Each contains three steel piles of cruciform section, driven to the rock.

The cruciform square-ended piles are built up of one 12-inch by 5-inch by 32-lb. joist, and two 6-inch by $4\frac{1}{2}$-inch by 20-lb. joists. The smaller sections are riveted to the larger with two lines of rivets, staggered at 7-inch centres. The actual weight of the piles with rivets is 77 lbs. per linear foot. The piles were driven with a McKiernan-Terry No. 9 steam-hammer giving 160 to 190 blows per minute with a steam-pressure of 95–100 lbs. per square inch. The final set was $\frac{1}{16}$ inch for 100 blows. The driving was very satisfactory: the piles went down without difficulty until the sandstone rock was reached, when they slowed up at once, entering the rock for a few feet. These piles were of various lengths in order to reach and enter the rock, the longest being 70 feet 4 inches, the shortest 52 feet 3 inches, and the average 61 feet 3 inches. The total load on one front cylinder is calculated to be 258 tons, or 86 tons per pile. The cylinders were filled with mass concrete for a height of 5 feet and then pumped out, the remainder of the mass-concrete filling being done in the dry.

The cylinders in the back row are 8 feet internal diameter and 6 inches thick. They are founded on gravel, and vary in height, the average being 13 feet 9 inches; the top is 2 inches above ordinary water-level as for the front cylinders. Each cylinder contains five steel piles, of similar section to those in the front cylinders, and also driven to the rock as already described. The cylinders were filled with concrete in a similar manner to the front ones.

Careful borings were made over the whole site of the wharf and transit-sheds. No extreme variation was found in the thickness of the various materials, which are shown in Figs. 3, 4, and 5. The rock was good, hard, red sandstone, underlying a thin layer of soft rock or

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compact red sand known locally as "roach." The tough brown clay contained a few stones, but not sufficient to be called boulder clay. The percentage of water in samples of clay taken from the front cylinder foundations was 14·36. The ballast varied in quality and was very sandy at about the centre of the wharf. The loamy sand was soft; in some places it was very fine and mixed with clay.

Pre-cast reinforced-concrete boxes are laid on top of the front and back cylinders (Figs. 6, 7, 8, and 9, Plate 4). Eight 1¼-inch diameter steel rods are bedded for a depth of 2 feet 6 inches in the top of the front cylinders and are surrounded with mass concrete, cylindrical in form, which ties the structure together. For the back cylinders a projecting cruciform pile serves a similar purpose.

Continuous longitudinal close timber fendering 6 inches thick is fastened by driving bolts to vertical timbers. The Author's experience is that close longitudinal fendering is the most satisfactory form of protection where the water-level is practically constant and where the principal wear on the timber is due to the ranging of the vessels moored alongside.

On both front and back pre-cast concrete boxes longitudinal reinforced-concrete ledges are formed to carry the pre-cast units forming the floor of the grain-subway, which is 15 feet 1 inch wide by 6 feet high. The floor is formed of pre-cast reinforced-concrete beams 40 inches by 12 inches in section, set close together with joggled joints which are filled with grout. The 3-inch mass-concrete floor is laid on the pre-cast beams, and covered with asphalt 1¼ inch thick, with a skirting 3 inches high along the walls. The roof of the subway also forms the deck of the wharf and is of considerable strength. Steel reinforcement left protruding from the front and back concrete boxes forms a bond with the roof steel, the whole being surrounded with concrete deposited in situ. The floor and roof of the subway were designed to sustain the blows of vessels coming alongside and to transmit the stresses to the structure behind.

The back concrete boxes are of simpler design than the front boxes as they contain no subway, but certain of the back boxes are of special design with chambers to house motors for driving the subway grain-belts. It is desirable to place the motors in compartments separated entirely from the subway for two reasons—the motors are kept free from grain-dust, and fire risk is eliminated. It is also desirable to arrange for the motors to be pipe-ventilated to the outer air. The front boxes weigh 67 tons, and the back boxes 62 tons each.

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A site on the bank of the Ship Canal was allocated as a block-making yard, and it was expected that the boxes would be set by means of the Canal Company's floating plant—a 60-ton crane and 250-ton sheer-legs. The contractors, however, elected to make the boxes on the site and set them by means of a specially-designed derrick with a lifting-capacity of 70 tons. This machine ran on the back row of boxes, with a leg on the first foundation block inside the transit-sheds. The scheme proved to be very satisfactory, as it is much better to set concrete blocks or boxes by means of fixed shore plant than by floating plant; owing to passing vessels the latter is always liable to be in movement, which makes accurate setting tedious and difficult. Although the first cost of the derrick was considerable, there is no doubt that the contractors were wise in adopting it, as they were able to make the concrete boxes on the site and set them continuously and expeditiously. The maximum number of boxes set in one week was six.

The rails for the two railways on the wharf are of tramway section, 126 lbs. to the yard, with a groove deep enough to take a wagon-wheel with standard flanges. This is a very good section for paved work as well as for wharves; it can be laid on cross sleepers and makes as cheap a railway as one laid with ordinary rails and guard-rails on longitudinal sleepers, and at the same time it is much more satisfactory. A further single rail is laid to form a 4-foot 8½-inch gauge railway with the front rail of the semi-portal crane, so that steam cranes can be brought on to the front of the wharf for special and heavy lifts.

The 153-foot length of wharf at the east end beyond the grain-subway, consisting of reinforced-concrete superstructure on concrete piles, calls for no special comment, except that it enabled interesting comparisons to be made between the concrete piles which were driven to the rock and those of cruciform section driven close to them. A bore-hole was specially put down close to the last cylinder, and the first three concrete piles in order to ascertain the exact level of the rock. The concrete piles were driven with a semi-automatic steam-hammer with a 3-ton monkey having a free drop of 3 feet 4 inches. The number of blows was about 40 per minute. The final penetration was ½ inch for 20 blows, and the points of the three cast-iron shoes all entered the rock to a depth of about 1 foot. The weight of each pile was 4·2 tons. The three steel cruciform piles in the adjoining cylinder foundation, driven by the McKiernan-Terry hammer with $\frac{1}{16}$ inch penetration for 100 blows, as already described, entered the rock to an average depth of 3 feet 8 inches. The weight of a pile was 2·1 tons. These

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results are satisfactory, and they prove that, with the final sets obtained, both the reinforced-concrete and the steel piles obtained a good bearing in the rock.

Constructional Difficulties.—The work in connection with the construction of the wharf was carried out satisfactorily; although the design was novel and difficulties arose during the progress of the work, they were all successfully overcome by the contractors. One of the principal difficulties arose in setting the concrete boxes on top of the concrete cylinders. The latter were specified to be levelled off to a true surface, and the concrete boxes were also to be true on their undersides; they were to be laid on the cylinders, the cylindrical space formed by two adjoining boxes being afterwards filled with concrete. The contractors formed the true surface on the concrete filling of the cylinders by first placing a steel sheath around the cylinders to keep the top free from water, and then levelling the mass concrete by screeding. It was not found possible, however, to make a sufficiently true surface or to make the underside of the boxes sufficiently true. This difficulty was overcome by forming a thin, flat cone of soft wood (yellow pine) 4 inches wide, which was cast with each box on its underside. When resting upon the cylinder the wood formed a packing around the outer edge of the cylinder and allowed for any inequalities in the bed; the whole space under the boxes on the top of the cylinder was filled solid with fine mass concrete when the cylindrical mass concrete was being deposited.

Part of the old wharf was constructed by driving whole-timber continuous sheeting at a batter of about 1 in 16 just inside the face-line; this sheeting held back part of the ground, as the slope was not fully excavated. The upper part of the wharf was constructed with piles and timber framing, and the sheeting was anchored with steel tie-rods to concrete blocks set well back in the solid ground. It was found, when commencing to set the front cylinders in front of the old timber sheeting, that the latter had been driven very irregularly and, in the case of eleven cylinders, fouled them at the bottom. This difficulty was overcome by cutting the cylinders at the back to clear the timber sheeting, placing concrete in bags at each side of the cylinder against the sheeting and then filling the cylinders with concrete to the top of the cut portion before pumping out to complete the filling in the dry.

In making the excavations in the slope for the back cylinders it was expected that the ground would be sufficiently stable to allow of the excavation being carried out with a steep slope at the back without disturbing the ground above water-level. A few foundations

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were successfully got out without the holes being of undue size, but it was soon found that the ground would not stand up and that the working-roads of the contractors and the ground up to and beyond the first row of foundation-blocks inside the transit-sheds would be disturbed unless some improved method were adopted. Steel interlocking sheeting, therefore, was driven on the arc of a circle of 7 feet 3 inches radius around the cylinder positions for half the circumference on the land side. Excavation was then carried out inside the sheeting to the foundation-level of the cylinder; the latter was set and concreted up, and the ground was filled in again against the sheeting before the latter was withdrawn to be used again. This solved the difficulty and reduced the excavation for each cylinder to reasonable proportions.

Fig. 3, Plate 4, shows that the existing slope extends under the site of the transit-sheds. It was therefore necessary to bridge this gap over, and the floor of the sheds for the first bay from the front is consequently of reinforced concrete, being part of the wharf structure. The concrete is of beam-and-slab form and was made in situ. It was originally intended to make it in separate pre-cast beams, but the contractors elected to form it in situ, and advantage was taken of this form of construction to connect the beams to the concrete boxes and the shed foundations, as there was some movement in some of the latter, which will be referred to later. A dwarf wall is formed between the shed-column foundations to carry the back of the floor between the foundations. This reinforced-concrete floor of the sheds is designed to carry a load of 5 cwt. per square foot.

Shed-Foundations.—All the shed-foundations previously constructed at Manchester Docks are of mass concrete, which is carried from the front to the back of the shed under each line of columns. The concrete is carried down to ballast and has been put in at the back of mass-concrete wharf-walls previously built; very little water has been met in the foundations, which have all been satisfactory. The loading at the bottom of the foundations is usually 3 tons and never more than 3·9 tons per square foot. This type of foundation was not considered suitable at Trafford wharf for several reasons: owing to the slope the concrete could be commenced only at the second line of columns from the front, and foundations of considerable area would be required, to spread the load; there was no wharf wall, and consequently the ground was full of water; the ballast was not good, being in some places nearly all sand; and with an open side towards the dock there was danger of forward movement or settlement. Piled foundations were therefore adopted.

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It was decided to construct the complete foundations for two sheds, in addition to the second row of column foundations which were necessary for the whole length to complete the wharf proper, as a first instalment of the wharf equipment.

Trial Piles.—In order to test the ground on the site of the transit-sheds arrangements were made to drive test piles. The site chosen was between the first two sheds to be built and on the line of the first row of columns inside the sheds next to the water front. Two reinforced-concrete piles, 34 feet 6 inches long by 14 inches square, with stock shoes having a 32-degree point, were driven by a 2½-ton semi-automatic steam-hammer supplied by the British Steel Piling Company, fixed on an ordinary pile-frame 40 feet high. The number of blows per minute was 30 to 40, except for the last 6 feet, when, owing to the breaking of the steam hose, the hammer was lifted by a steam winch, the number of blows per minute being reduced to 10. The drop was 3 feet, and a hitchpin dolly 4 feet long was used. The top of the dolly was fitted with an iron ring; the bottom was un protected, but was surrounded with a wooden frame reinforced with steel bands. Sawdust or wood pulp was placed on the pile, but no steel helmet was used. The dollies frequently wore down and were cut until they were reduced to 2 feet in length, when they were discarded; five dollies were used for the two piles. The two piles were driven to final sets of 1½ inch and 1⅝ inch for 100 blows, or an average of $\frac{1}{64}$ inch per blow, the toe of the piles being at about the level of the canal-bottom. The piles when driven were 32 feet into the ground, of which 12 feet was in the underlying clay. The safe load which these piles would sustain, according to the Wellington or *Engineering News* formula,[1] is 130 tons. Details of the driving and the strata passed through are given in *Fig. 10*.

As the piles were practically driven to refusal, it was decided to adopt piles 35 feet long for the shed-foundations and to drive them to at least 1 foot below canal-bottom; the heads would then be 5 feet above the bottom of the mass-concrete foundations, allowing a small margin if it were found necessary to drive the piles deeper. A clause was inserted in the specification to ensure results similar to, or better than, those which had been found to be satisfactory when

[1] The formula is as follows:—

$$L = \frac{2WH}{S + 0{\cdot}10}$$

Where W denotes the weight of the hammer in tons,
H „ „ drop in feet,
S „ „ final set in inches,
L „ „ safe load in tons.

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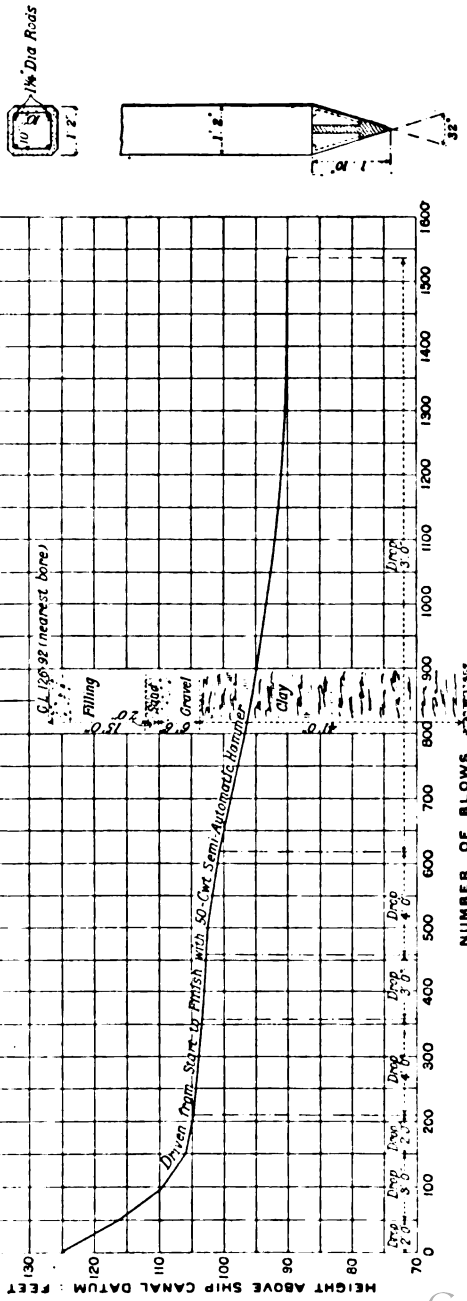

Fig. 10.

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driving the trial piles, since it was expected from the trial that some difficulty would be encountered in driving to the specified depth. The piles were, therefore, specified to be driven to a final set of $\frac{3}{8}$ inch for 20 blows, or $\frac{3}{160}$ inch per blow, with a 2½-ton hammer dropping 4 feet. The extra drop was specified to ensure the piles being driven home if the ground proved to be more difficult than that encountered at the trial.

Reinforced-Concrete Piles.—The piles are 35 feet long, of octagonal shape, 15 inches over the flats, the sectional area being 186 square inches and the perimeter 49½ inches. The reinforcement consists of eight 1-inch diameter rods with a ⅜-inch diameter spiral at 2 inches pitch. The shoe is of chilled cast iron with a long taper, octagonal in shape and having an angle of 22 degrees. The steel reinforcement amounts to 28 lbs. per linear foot of pile, or 4·38 per cent. of the volume. The total weight of a 35-foot pile is 3 tons.

Weights on Foundations.—The weights on the foundations at the level of the bottom of the mass-concrete foundation-blocks, as given in Fig. 3, Plate 4, include the structure, crane loads, and loads on floors. The loading on upper floors adopted for all Manchester Ship Canal transit-sheds is 30 cwt. per square yard, or 3⅓ cwt. per square foot. It has been found where there are several floors that, allowing for loose stacking and gangways, this load cannot be stored in practice on all floors, and a reduction of 33 per cent. is taken from the maximum load on the floors when calculating weights on the foundations of a large building. For small buildings or single floors the whole theoretical load on the floors should be taken.

Design of Shed-Foundations.—The columns of the buildings (except the front line carried on cylinders and steel piles as already described) are on mass-concrete foundations carried on piles, the number of piles being proportioned according to the weight to be supported. It was originally intended to make the mass-concrete foundations for the inner columns of the building 10 feet by 9 feet 6 inches, each being carried on nine piles. Each pile, therefore, would sustain a load of 66 tons, without making any allowance for the support afforded by the ground beneath the mass-concrete foundations. In the outer or south-wall foundations, which have smaller weights to carry, the number of piles was reduced to five, each with a load of 66·4 tons. In the row of columns along the face of the sheds, which are carried on cylinders and on five cruciform piles driven to the rock, each pile sustains a load of 108 tons, not allowing for any part of the load being carried by the ballast on which the cylinders are founded.

Pile-Driving.—The contractors commenced by excavating a hole

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down to approximately 1 foot above the finished level of the excavation for the mass-concrete foundations, and then pitched the piles by means of a crane, the piles being spaced by a whole-timber frame fixed over the excavation. The piles were partly driven at first by a McKiernan-Terry double-acting steam-hammer No. 9 B, but after penetrating the clay the piles pulled up, the penetration being only about 1 inch for 100 blows. The McKiernan-Terry hammer, which is bolted to the top of the pile and weighs 6,800 lbs., contains a double-acting piston, 8½ inches in diameter, with a stroke of 16 inches, and an anvil block transmits the blow to the pile. The average number of blows was 110 to 120 per minute, with a boiler-pressure of 95–100 lbs. per square inch. There appears to be no reliable formula giving the safe load of a pile driven by this type of steam-hammer, but it is interesting to note that piles which finished at 1 inch to 100 blows under this hammer continued to go down at the rate of 12 inches for 20 blows when driven by a 3-ton semi-automatic hammer falling 3 feet 4 inches. The semi-automatic hammer was also fitted on the top of the pile and was steadied during driving by being hung from a crane. The type of hammer used had a maximum stroke of 4 feet 6 inches with a 3-ton monkey lifted by steam and dropping freely when released. The usual number of blows given per minute was about 30. The partial driving by the McKiernan-Terry hammer was soon abandoned, owing to the time lost in driving and changing hammers, and also to the much greater consumption of steam compared with the semi-automatic hammer. The piles were protected from damage by a hardwood packing 6½ inches thick by 11½ inches diameter, with hooped ends. Oak and hickory were tried, but elm was found to be most satisfactory. In addition coarse sacking, which frequently caught fire, was placed on the pile.

Of the first piles moulded, 374 were 35 feet long, and 200 were 33 feet 6 inches long. These dimensions were determined on the data gained from driving the trial piles. The actual sets obtained were actually much in excess of those obtained at the trial, the average being about 4 inches for 20 blows. The piles were driven 2 feet deeper than was shown on the contract drawings, or to within 3 feet of the bottom of the excavation for the mass-concrete foundations, the shoes of the piles being about 4 feet below the level to which the test piles had been driven. These sets were not satisfactory, the safe load by the Wellington formula being only 67 tons instead of 130 tons, the trial-pile calculation.

Seventy-seven further test piles were then driven over the whole site; the sets on these were very diverse, ranging between 3½ and

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7 inches for 20 blows. It was further decided to mould the rest of the piles to a length of 40 feet, and when the first of these had matured, six further test piles were driven, the final sets being rather worse than before, ranging from $5\frac{3}{4}$ to $9\frac{3}{4}$ inches. As an experiment one of the original trial piles was re-driven with the 3-ton semi-automatic hammer, but with a $2\frac{1}{2}$-foot drop, in order to reproduce the former conditions as nearly as possible, but without the 4-foot dolly. The result was as follows :—

First	20 blows	No movement.
Second	„	$\frac{1}{2}$ inch set.
Third	„	$1\frac{1}{4}$ „
Fourth	„	$\frac{7}{8}$ „
Fifth	„	1 „

Thus the average set during the re-driving was $\frac{1}{20}$ inch, compared with $\frac{1}{64}$ inch per blow 2 years previously. Probably increased sets would have been obtained if the second driving had continued, although not so great as those obtained in driving the new piles.

There is no reason to believe that the trial piles were given less than the 3-foot drop, since they were carefully inspected. The explanation of the difference lies apparently in the use of the 4-foot dolly, which must have had a very large cushioning effect.

There appeared to be no explanation of the very variable sets obtained over the whole site with the test piles, the material taken from the bore-holes being, on the whole, of the same consistency and from similar strata, and there being no great variation in the depth to which the piles penetrated the clay. As the final sets had been so variable, it was decided to make a further test by loading an actual group of foundation piles to considerably more than their working load. The group selected was remote from the area where the contractors were at work, being an outside group of five piles at the east end, which had all been driven to a set of 3·8 inches for 20 blows. Four of the piles were 35 feet long, and the remaining one 33 feet 6 inches. The points of the four piles were 4 feet below the canal-bottom ; and the strata, commencing at the bottom, were : 9 feet of clay, 13 feet of ballast, 6 feet of sandy loam, and 4 feet of filling. A reinforced-concrete platform was built on the piles clear of the ground, in order to give a 14-feet-square base on which steel billets could be stacked. Loading commenced 34 days after the piles were driven, and was carried on regularly over a period of 11 days. The full load was kept on for 5 days and taken off in 3 days. The full load on the five piles, including the concrete platform, was 412 tons, or 82·4 tons per pile. The load to be

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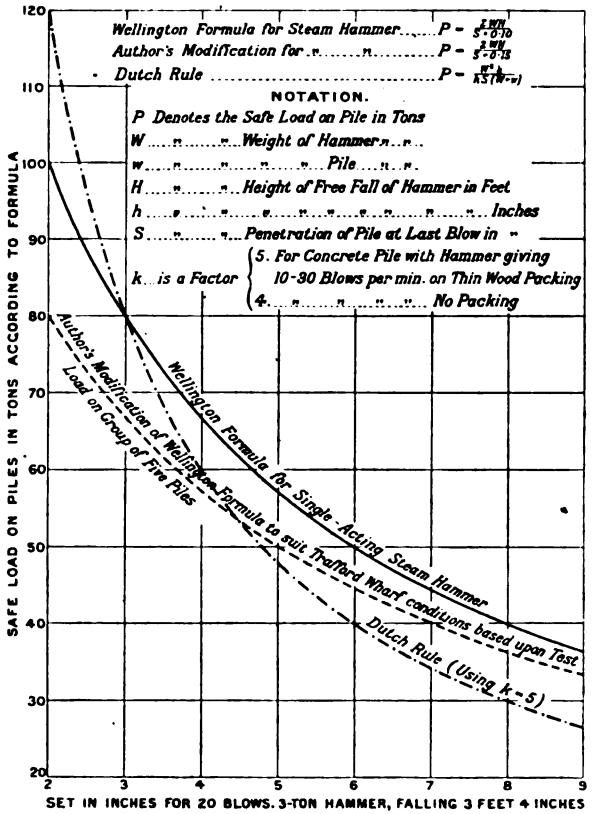
sustained by this foundation is 300 tons, or 60 tons per pile. The maximum settlement was $\frac{5}{8}$ inch, of which $\frac{3}{16}$ inch took place when 300 tons (or 60 tons per pile) had been put on and the remainder gradually, as the loading continued; the settlement then remained constant until the removal of the billets. The piles recovered $\frac{1}{4}$ inch as soon as the load was removed, and were within $\frac{1}{8}$ inch of their original level after $10\frac{1}{2}$ months. From this experiment it was assumed that

Fig. 11.


Wellington Formula for Steam Hammer ..... P = 2WH / S+0·10
Author's Modification for " ..... P = 2WH / S+0·15
Dutch Rule ..... P = W²h / KS(W+w)
NOTATION.
P Denotes the Safe Load on Pile in Tons
W " " Weight of Hammer "
w " " " Pile "
H " " Height of Free Fall of Hammer in Feet
h " " " " " " Inches
S " " Penetration of Pile at Last Blow in "
k is a Factor 5. For Concrete Pile with Hammer giving 10-30 Blows per min. on Thin Wood Packing
4 " " " No Packing
Author's Modification of Wellington Formula to suit Trafford Wharf conditions based upon Test Load on Group of Five Piles
Wellington Formula for Single-Acting Steam Hammer
Dutch Rule (Using k = 5)
SAFE LOAD ON PILES IN TONS ACCORDING TO FORMULA
120
110
100
90
80
70
60
50
40
30
20
2
3
4
5
6
7
8
9
SET IN INCHES FOR 20 BLOWS. 3-TON HAMMER, FALLING 3 FEET 4 INCHES

piles with a 3·8-inch set for 20 blows, driven under the conditions described, namely, by a 3-ton semi-automatic hammer with 3 feet 4 inches drop, could safely sustain a load of 60 tons without undue settlement. The settlement under the 60-ton load per pile was practically negligible, and the settlement under the increase of load beyond this up to 82·4 tons per pile was very little and was directly proportional to the increase of the load. *Fig. 11* gives the

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curve of safe loads as calculated by the Wellington formula for various sets with the 3-ton hammer and a 3-foot 4-inch drop. The second curve on the diagram (shown dotted) has been obtained by increasing the lower factor from 0·10 to 0·15, which gives a safe load of 60 tons for a set of 3·7 inches for 20 blows, a near approximation to the safe load on the piles as found by experiment. The reduction in the curve of safe loads as is shown on the diagram (*Fig. 11*), appears to be a wise precaution, and it is the intention of the Author to use the modified formula when conditions warrant it. The curve given by the Dutch rule is plotted in the same diagram as a matter of interest.

The splicing of piles is always unsatisfactory and slow, entailing considerable extra expense and delay. It was thought to be unadvisable to mould the remaining piles to a greater length than 40 feet, since, if any slight settlement is to take place, it is better for it to be of an even and regular character, and in any case it was doubtful whether good sets would be obtained without driving to the rock. It was therefore decided to increase the number of piles in each foundation, so that the load would be safely sustained according to the revised formula. The numbers of piles in the various foundations were decided upon according to the test piles. On the whole the increases were from nine to twelve piles in the different foundations over the area of one shed, with additional piles in certain foundations of the other shed which have specially high sets.

The load to be carried on the groups of piles under the columns of the buildings is 594 tons. With a group of nine piles, as originally designed, the load per pile is 66 tons—according to the modified formula the final set should be not more than 3·1 inches for 20 blows. If the number of piles is increased to twelve, the load per pile is reduced to 49·5 tons, and the final set should be not more than 5 inches for 20 blows. Fig. 2, Plate 1, indicates the number of piles driven in various foundations. The number in each group was decided from the information gained from the driving of the seventy-seven test piles over the whole area. The foundations for the first row of columns inside the sheds at the back of the wharf have a minimum of twelve piles; the rest of the inner column foundations have, in general, nine or twelve piles, except that in some cases the number has been increased beyond twelve where the sets were higher than the average.

Form of Shoe.—As already stated, the contract form of shoe had an octagonal chilled cast-iron point with an angle of 22 degrees. Experiments were made with other points to determine whether the fine point accounted in any way for the greater penetration

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obtained in practice, compared with that obtained in driving the original test piles.

The tests made were as follows :—

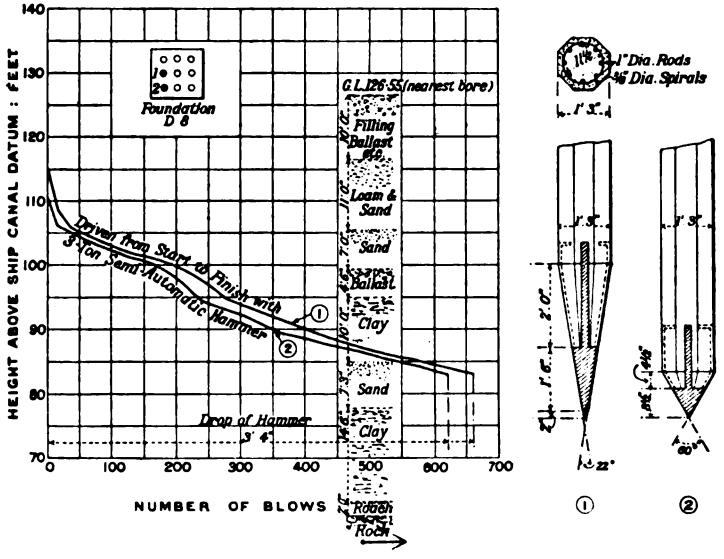
Experiment A.

(1) 15-inch octagonal pile 40 feet long with contract 22-degree shoe.

(2) 15-inch octagonal pile 40 feet long with 60-degree cast-iron shoe.

The two piles were driven close together in the same group (*Fig. 12*). The successive degrees of penetration throughout the process of

Figs. 12



driving were practically identical, and the final sets were also similar, being 6½ inches for 20 blows for No. 1, and 6 inches for No. 2. Blunt points, (No. 2 in *Figs. 12*) were fitted to 294 piles, but there was no appreciable difference between the results of the driving of these piles and of the contract piles.

Experiment B.

(1) 15-inch octagonal pile 40 feet long with contract 22-degree shoe.

(2) 15-inch octagonal pile 40 feet long with shoe removed, leaving flat bottom 8 inches over flats.

The two piles were driven at opposite corners of the same foundation. Records of driving are shown in *Fig. 13* and are not dissimilar.

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The contract pile (No. 1) finished at 4½ inches for 20 blows, and the flat-nosed (No. 2, *Figs. 13*), contrary to expectation, finished at 6 inches for 20 blows.

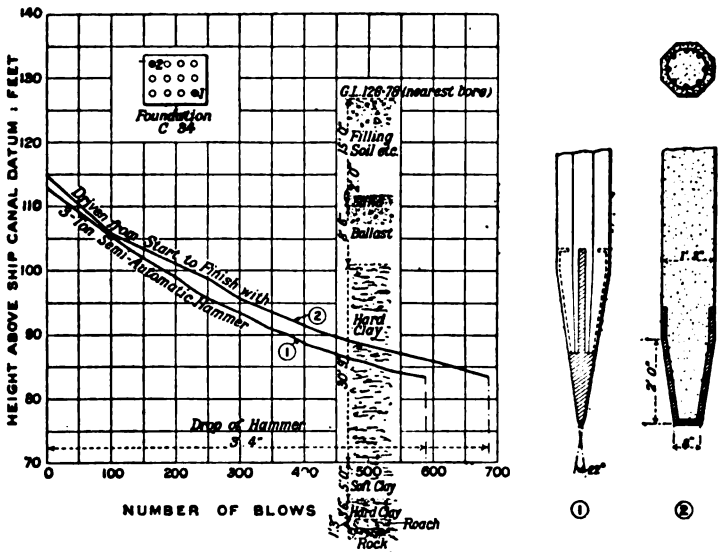
Experiment C.

(1) 15-inch octagonal pile 38 feet 6 inches long with contract 22-degree shoe.

(2) 15-inch octagonal pile 35 feet long with contract shoe removed and point cut down to 12 inches over flats.

The two piles were driven at opposite corners of the same founda-

Figs. 13.



tion (*Fig. 14*). The penetration during driving was again very similar, pile No. 1 finishing with a 6-inch set for 20 blows, and No. 2 with a 7-inch set.

The explanation of this unexpected result appears to be that the flat form of end opens the strata out slightly more than does the pointed end and decreases the skin friction during the process of driving.

Double Timber Piles.—Some of the piles (24 inches by 12 inches in section) removed from the old timber wharf were used under the dwarf walls between the foundations for the first row of columns at the front of the transit-sheds. They were driven by the same semi-automatic steam-hammer as was used for the concrete piles, and the conditions of driving were identical. They were in 14 feet of ballast and penetrated the clay to a depth of 7 to 8 feet. The

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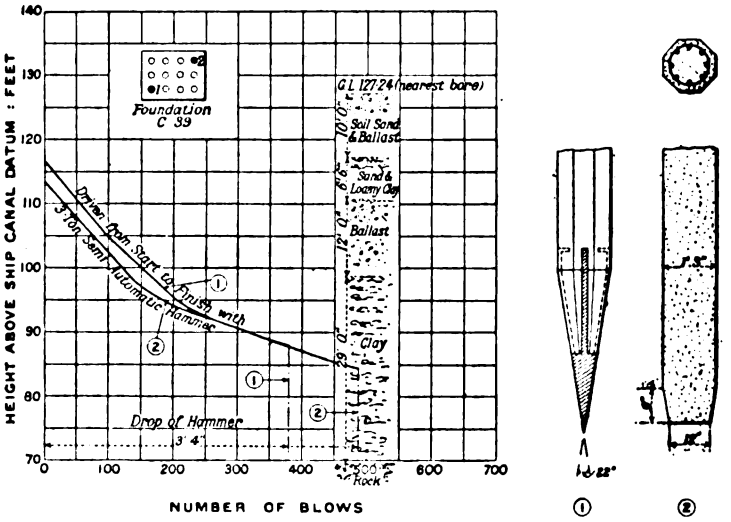
piles having rock pile-shoes finished with a 3½-inch set for 20 blows, and those with an ordinary type of shoe, with a 1½-inch set. The larger sectional area of pile as compared with the octagonal contract piles appears to have improved the set.

Varying Sets.—The final sets of various groups of piles quite close together varied very considerably. For instance, three contiguous groups of piles, 40 feet long and driven to the same level, had average sets of 5·23, 2·71, and 4·92 inches for 20 blows. The middle group was 25 feet between centres from the other two.

Group Driving.—On the whole, group driving gave better sets than single driving; for example, at a foundation with twelve piles all 40 feet long, a trial pile at the centre of the foundation was first

Fig. 14.



driven and gave a set of 7 inches for 20 blows. The remainder were driven a fortnight later. They were all pitched together and driven in turn so that no pile was at any time more than 10 feet lower than the others. The average set for 20 blows for the whole group worked out at 3·95 inches. The piles were 2 feet 9 inches apart between centres.

Order of Driving.—The order of driving or the position of the piles in a group did not appear to be a determining factor in the set. Sometimes the first pile would give a better set than the last pile in a group, and there were many examples of central piles which were driven last finishing with a worse set than corner piles in the same group.

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Redriving.—Some experiments were made in redriving, with very satisfactory results, proving that the frictional resistance of the piles increased very considerably in a comparatively short period. The piles were all 40 feet long and driven to the same level. The final sets for 20 blows with a 3-ton hammer falling freely 3 feet 4 inches were as follows :—

Set at End of Continuous Driving.	Interval.	Set on Resumption of Driving.
Inches.	Months.	Inches.
6	8	2½
3½	8	3
5½	7	1½
7½	5½	2
3¾	10½	2
4	12	1½
4	12	1½
4	3	3
5¾	3 days	4½

The number of piles for the whole scheme as originally designed was 1,685. This was increased by 506 to 2,191.

Movement of Foundations.—During the progress of the work it was found that the mass-concrete foundation-blocks for the front row of shed columns for the upstream of the two transit-sheds were moving slightly towards the canal. The face of these blocks is only about 9 feet away from the water's edge. The mass concrete weighed about 77 tons, and there were twelve piles in each of the foundations. The total movement ranged from 2¼ to 5 inches, spread over a period of 6 months. A greater thickness of ballast and sand overlies the clay at this part of the frontage, and both the sand and ballast are finer than elsewhere. No tilting of the blocks could be detected by measurement, but as the piles are embedded in the underlying clay to a depth of about 18 feet, there may have been some very slight tilting, as there is no reason to believe that the clay had moved forward and taken the whole work with it. No doubt the work of excavating the slope for the two rows of cylinders was the cause of some movement in the fine sand, and the ground in moving forward took the foundation-blocks with it; the

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steel sheeting half surrounding the back cylinders stopped any serious movement. The foundations which moved were all anchored back first to the foundations immediately behind them, which in their turn were also tied to the next foundations in the rear. The ties to each foundation consisted of three 2-inch diameter rods embedded in concrete with their ends bonded into the mass concrete of the foundations. No further movement has been observed since these ties were completed: perhaps it would have ceased in any case after the wharf had been completed and had become a homogeneous structure; it was necessary, however, to stop movement during construction, and this was effectively accomplished by the steel ties.

Notes on Contractors' Plant.—The contractors used steel forms very extensively. Simple forms were used for the reinforced-concrete cylinders and also for the more complicated reinforced-concrete boxes. The latter forms were not sufficiently stiff at first and there was some bulging of the boxes, but this was overcome by additional ties and timber strutting. Steel forms were also used for the underside of the decking, both over the subway and between the back concrete boxes and the first line of columns. The forms were lowered, run along under the deck, and lifted to their new position with the minimum of disturbance.

Steel forms impart an excellent surface to the concrete, and their adoption is certainly worth while from a contractor's point of view if the design is fairly simple and there is sufficient repetition. Carpenters to-day are hard to obtain, and timber is expensive. Steel forms have recently been used on another canal contract for reinforced-concrete piles. The piles have been of good shape, with clean faces, and from the contractor's point of view have no doubt been a success.

Pile-Making.—Owing to the cost of making an extensive block-yard, a contractor is always reluctant to provide adequate floor-space and sufficient moulds to allow the piles to mature on the floors before lifting. For this contract the minimum time specified before lifting was 28 days. The contractors wished to have this period reduced, and, in order to be able to lift the piles without stressing the setting concrete, a rigid frame with a five-point lift was devised. This frame was 25 feet long, which left 7 feet 6 inches at each end of a 40-foot pile unsupported, the points of suspension being 6 feet 3 inches apart between centres. The frame was sufficiently rigid to obviate undue stresses in the piles during lifting and stacking. Permission was given for piles to be lifted with this device from the floor to the stack-yard in 14 days. This

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treatment did not appear to damage the piles, as only three piles had to be drawn after partial driving owing to cracking; these cases could not, however, have been caused by premature lifting.

It is of interest to note that one pile which had to be extracted, after having been driven 20 feet into the ground, was drawn by rope tackle and six-purchase blocks. The dead pull required to start the pile was calculated to be 100 tons.

Concrete.—The mass concrete in foundations, the concrete boxes, and the reinforced-concrete decking formed in situ, were all mixed in two ½-cubic-yard concrete-mixers and delivered through an inclined pipe from a tower mounted on trucks running the whole length of the wharf, and of sufficient height to command the full width of the site.

The proportions of materials forming the various concretes were as follows:—

	Aggregate.	Sand.	Cement.
	Cubic Feet.	Cubic Feet.	Pounds.
Piles	27	13½	810
Reinforced-concrete cylinders	27	13½	810
Reinforced-concrete boxes, decking, floor-beams, subway-beams, top 2 feet of cylinder filling	27	13½	728
Mass concrete in shed-foundations and cylinder-filling (all but 2 feet at top)	27	13½	610

Transit-Sheds.—The erection of the transit-sheds is in abeyance at the present time, the Canal Company having decided to erect somewhat similar sheds elsewhere at the Manchester Docks before starting those at Trafford wharf.

Cost.—Unit costs in connection with this work are not very useful, as it was commenced in 1921, when prices were abnormally high, and also because the wharf is of a special type and cannot be separated from the foundations of the sheds.

The total cost of the 1,653 feet of wharf and foundations of two sheds, including the railways on the wharf, amounted to £335,163.

The cost of the wharf and railways, including the two lines of cylinders, the foundations for the first line of columns, and the shed floor to this line, amounted to £168 per linear foot.

The cost of the type of wharf at the east end, constructed of reinforced-concrete piles and superstructure, inclusive of railways, amounted to £71 per linear foot.

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The work was designed by, and carried out under the direction of, the Author. The drawings were prepared by the late Mr. W. Johnson. The Resident Engineer was Mr. G. Leader Williams, and the Assistant Resident Engineer Mr. H. K. Russell, Assoc. M. Inst. C.E., to whom the Author is indebted for his assistance in the preparation of the details for this Paper.

The Contractors were Messrs. Kirk and Randall, Ltd., their Chief Engineer being Mr. F. W. Duckham, M. Inst. C.E., and their Agent in charge of the work Mr. John F. Matthews.

The Paper is accompanied by seven sheets of drawings, from some of which Plate 4 and the Figures in the text have been prepared.

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Discussion.

Mr. H. J. DEANE considered that one of the most useful parts of the Paper concerned the very interesting tests on piles. He had not seen the results of any tests carried out quite on such lines before, and the information would be of great value for future reference. The Author suggested that it was not within the scope of the Paper to give the result of the consideration of the question of the design of the wharfing itself. He thought the Author had thereby withheld valuable information, because the consideration of a design of that kind must have been of great interest to those concerned, particularly as it had resulted in a scheme which was, in his experience, unique. The fact that the pre-cast boxes weighed 67 and 62 tons indicated a work of some magnitude—which was emphasized by the costs given on p. 87. The Author stated that the cost of the heavy part of the wharf was £168 per linear foot, which was equivalent to about £3 per square foot of surface, taking a 57-foot width from the front of the quay to the second row of columns. That figure seemed to be extraordinarily high, particularly when it was realized that every 2*s.* per square foot represented £10,000 in the capital outlay of the undertaking. The cost was apparently mainly due to the intricate concrete work and the powerful tackle which had had to be used in handling the heavy weights. Had it been necessary to build those boxes up in such a form? Would it not have been possible to build them 25 feet long, but of less weight, so that they could have been handled much more expeditiously and at considerably less cost? It was impossible to install heavy lifting-appliances, such as had been used here, without adding very materially to the contractors' cost. The Manchester Ship-Canal apparently had two very effective floating derricks, one capable of lifting 60 tons, and the other 250 tons. It seemed to him that, if the contractors had been given suitable terms for the use of that plant, they would have used it in preference to the 70-ton derrick which was specially constructed. He did not suggest that the work would have been executed as satisfactorily; but, looking at the matter from the point of view of cost, he felt sure that the contractors would have preferred to do the work in the cheapest way possible, provided they could execute it efficiently. In his own experience floating derricks and heavy lifting-appliances held by Port Authorities were generally charged for at rates which Mr. Deane

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Mr. Deane. were commensurate with their value for commercial purposes. An engineering work could not be regarded as a commercial work in the same sense. The traffic departments of port authorities reaped the benefit of the charges for the services of those tools; and, where a job was being done by the engineering department, the latter had to pay the contractors for the use of such appliances, and the traffic department reaped the benefit. In other words, it was simply a case of robbing Peter to pay Paul, and the authority was neither better nor worse off. In the present case the plant had been already available, but the contractors for some reason—presumably because the charges were too high—had elected to spend several thousand pounds on a 70-ton derrick. The result was that the engineering department of the Manchester Ship-Canal had had to pay probably for the whole of that plant, less its scrap value, and the traffic department had got nothing; so that economically the authority had paid more than they would have done if the charges for the floating derrick had been reduced to a reasonable rate.

Mr. Wilson. Mr. M. F.-G. WILSON agreed with Mr. Deane that very valuable information had been given in the Paper in connection with pile-driving. The design of the wharf was remarkable for the great strength and massiveness of the different parts. He was not sure that he agreed with Mr. Deane in his explanation as to why floating cranes had not been used. In the Paper it was stated that they were not used on account of difficulty in setting the boxes, chiefly because of the movement of the water which was caused by passing ships. He knew from personal experience the extreme difficulty and the delay that occurred in trying to set work accurately from a floating machine which was constantly on the move. A fixed derrick on land could be kept under control and used with much more effect. The information which the Author gave with regard to pile foundations was exceedingly valuable. There were, however, matters in connection with the loading which were not quite clear. Experiments had first been made with some test piles which were calculated to carry 130 tons as a safe load; but, apparently owing to the use of a dolly, it had not been found possible to get the same result when the main piles were driven. What exactly was the final figure taken as a set for the main piles? Two figures were given: one for a load of 66 tons, which was eventually brought down to 49·5 tons; and a formula was given which apparently suited the latter weight of 49·5 tons per pile. Was that the one which was actually adopted? Nor was he quite sure why the Author had considered it necessary to modify the Wellington formula; the first piles driven would, according to the formula, carry a load of 60 tons with safety, and that was the load which was required at that time; but eventually,

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by adding more piles, the load was reduced to 49·5 tons. He considered that, if the formula gave the safe load per pile as 60 tons, it was not necessary to alter it for a load of only 49·5 tons. He might have misunderstood the argument, but he would like to know more clearly why that had been done. The Author stated that the McKiernan-Terry pile-hammer had been satisfactory when used for driving steel piles into the rock, but that later, when it was used for driving the reinforced-concrete piles, after driving to a set of 100 blows per inch on a pile, it had been found that when an ordinary free-falling monkey was applied the piles continued to drive quite easily for several feet deeper. The Author suggested that it was a pity there was no reliable formula for such pile-engines. Mr. Wilson's firm had had exactly the same experience some years ago with a somewhat similar type of automatic pile-engine, which had been used on some bearing-piles required to carry a crane-road. One pile had been driven during a whole day with that machine with very little result. Then an ordinary free-falling monkey was used, which drove the pile down another 8 or 10 feet without the slightest difficulty. It was obvious that those light, rapid-action pile-engines were of no value for bearing-piles, and therefore he did not see any use in providing a formula which would suit them; the best course was not to use them. On the question of the pointing or otherwise of piles, in Trautwine's pocket-book it was stated that some square-ended piles had been found to drive more easily than pointed piles. Mr. A. S. E. Ackermann, Assoc. M. Inst. C.E., had described a number of experiments which he had made on small piles driven into clay.[1] He first showed from theoretical considerations that the pointed pile ought to carry more weight than the blunt pile, which deduction he proved by experiment. This was now supported by the Author's full-scale experiment with piles driven into clay. The proportions of the concrete were given in pounds of cement and cubic feet of sand per cubic yard of aggregate, which was rather confusing. The concrete for the piles appeared to be a 1 : 1½ : 3 mixture, and the mass concrete a 1 : 2 : 4 mixture, the composition of the boxes having an intermediate value. He would have thought that the concrete for the boxes might quite safely have been a 1 : 2 : 4 mixture, which was a very suitable proportion, especially with cement of the excellent quality that was obtainable at the present time.

Mr. Wilson.

Mr. Gerald Fitz-Gibbon remarked that the Paper was particularly interesting because the work described departed from the usual conventional style of wharf-construction. He had been engaged for

Mr. Fitz-Gibbon.

[1] Trans. Soc. Engineers, 1919, p. 37.

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Mr. Fitz-Gibbon.

many years on the construction of the Manchester Ship-Canal and therefore the Paper was interesting to him, as being brought forward by the Engineer of that canal. Having regard to the great variety and the importance and magnitude of the works on the canal, and the short time in which they had been carried out, he considered that it was the finest engineering work in the world. It was therefore only fitting that an engineer of that undertaking should describe a novelty in wharf-construction which added to the list of important works that already stood to the credit of his predecessors. He did not think that the derrick crane used for placing the boxes cost many thousand pounds, and it represented an insignificant portion of the cost of the work. He was certain that, unless the contractors had seen that it would facilitate their work to have the derrick on solid ground, they would have made use of the floating crane. He would like further information about the manner in which the pre-cast boxes had been connected at their ends. He could not see that there was any projecting steelwork at the ends, and it appeared possible that, under the action of heavy loads and vibration set up by the travelling cranes, and in the absence of any connection between the several boxes, cracks might have appeared already at the junctions. He was not in favour of the use of a dolly of any sort, or of any cushion or helmet interposed between the monkey and the pile-head. His firm were engaged some years ago upon the widening of the Admiralty Pier, Dover, and a large number of reinforced-concrete piles, up to 75 feet long, were driven in the course of that work through about 40 feet of chalk filling and 9 to 15 feet into solid chalk. The longest piles weighed about 8½ tons each, and they were driven by a 4-ton ram with a fall of about 4 feet. Nothing was placed on the head of the concrete pile, and during the whole of the work none of the pile-heads was injured by the driving. Energy must be absorbed by a dolly, helmet, or cushion, and the results given in the Paper seemed to support that view. Some form of dolly or packing had apparently been used for the test piles. He imagined, too, that the Wellington formula, and the Author's modification, took into consideration the use of the dolly or packing. He had applied to the data given in the Paper a formula brought out nearly 40 years ago by his late partner, Mr. A. C. Hurtzig, M. Inst. C.E., and constantly and successfully used by his firm. A slight modification had been necessary, because the formula had been devised before reinforced-concrete piles were used. Using that modified formula, the extreme resistance of each pile in the group of five piles was 85·65 tons. The Author stated that the piles were loaded to 82·4 tons, that

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up to that point there was a very slight settlement, but at that point it ceased, and there was practically no further movement. That showed that Mr. Hurtzig's formula must be very nearly correct. The Wellington formula, taking the same data, gave 68·96 tons. The Author's modified Wellington formula gave 58·82 tons, and he thought the Author was right in stating that the safe load would be 60 tons. The permissible maximum load on such a pile, taking the extreme stress in simple compression as 600 lbs. per square inch for concrete, and 9,000 lbs. for steel, was 73·38 tons. Mr. Hurtzig's formula had been widely adopted, and it was a great pleasure to Mr. Fitz-Gibbon to find, on comparing it with the more modern formulas, that it was, if anything, more correct. He had alluded to the magnitude and variety of the works on the Manchester Ship-Canal, and he would recommend those of the younger members who did not know much about those works to study the excellent Papers in the "Proceedings"[1] on the subject. Mr. Fitz-Gibbon.

Mr. A. H. ROBERTS considered that the design of the wharf described afforded a good example of the usefulness and adaptability of reinforced concrete to various problems. The method of introducing a grain-subway by means of pre-cast boxes was ingenious and suitable. He had always preferred to have overhead grain-belts; he did not like belts in tunnels, though there were tunnels at Manchester. The Author had overcome the difficulty in the case of these tunnel boxes, because they were below quay-level and at the same time they were above ground-level and water-level; so the Author had done what seemed almost impossible. The only likelihood of any trouble with those subways was at the joints, and, no doubt, no difficulty would arise in practice, because there was no head to drive the water through. He observed that the cruciform steel piles had been driven 3 feet 8 inches into hard sandstone rock; it would be interesting to know how far a reinforced-concrete pile would penetrate some kind of standard sandstone. The variation in the sets of the piles at different parts of the area in similar strata was remarkable, but it was what all engineers met with. Still more remarkable were the varying sets obtained in the same group of piles. The Author suggested that in the case of the conditions at Manchester the factor of 0·1 in the Wellington formula should be increased to 0·15. Mr. Roberts noticed that Mr. C. R. Gow suggested[2] that that factor should not be a constant, but should Mr. Roberts.

[1] Minutes of Proceedings Inst. C.E., vol. cxxxi (1897), p. 14.

[2] F. W. Taylor, S. E. Thomson, and E. Smulski, "Concrete Plain and Reinforced," p. 547. New York, 1925.

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Mr. Roberts

take account of the effect of the relative weights of the pile and the hammer, and that the weight of the hammer divided by ten times the weight of the pile should be substituted for the factor 0·1. With a 3-ton pile and a 3-ton hammer that gave a factor of 0·1; but any variation in the relative weights of the pile and the hammer would introduce a corresponding variation of the factor. He had recently made and driven 692 reinforced-concrete piles for foundations on an area only 150 feet by 114 feet. Piles driven in groups of four or five had not shown the anticipated tendency to drive increasingly stiffly; the last pile had driven about as easily as the first. The specified set at the start was intended to be 0·5 inch to ten blows, but it had been found, after getting 1 inch to ten blows, that the hammering was doing no good. Therefore the 1 inch to ten blows had been adopted throughout. The contractors seemed to have used an elm dolly in the case of the Author's work. In his own case they had started with a greenheart dolly, but that had been found to be much too hard. They had then tried an elm dolly, and their experience had been that that went to pulp after driving two piles, due to the fact, they thought, that there was always hot water, from condensed steam, coming down the piston from the hammer. They had then substituted home-grown oak, which had given entire satisfaction, twenty or thirty piles being driven with every dolly. They had expected, when they started, that the driving of that number of piles in a limited area would have the effect of raising the surface of the ground about 12 inches, and they removed the surface to that depth. What actually happened was that the entire surface went down 12 inches, and they had to make it up. The explanation obviously was, although it had been contrary to expectation, that consolidation had taken place not only horizontally alike in all directions, but also vertically in the direction in which the piles were driven and in which gravity was operating. The sets obtained at Trafford wharf after redriving were satisfactory and confirmed general experience. It was not often that the opportunity arose of redriving and of noting the result. He thought the information obtained by the test loading of the piles was also extremely valuable. Had the Author used at Manchester asphalt on concrete for the flooring of sheds, and, if so, had it been in use for some years. Further, was there a tendency for the asphalt to creep along in waves under traffic?

Mr Cleaver.

Mr. WILLIAM CLEAVER remarked that the Author had utilized a principle which he himself had advocated for many years. He believed he had been one of the first, if not the first, both to advocate and to adopt pre-cast work on reinforced-concrete jetties and other

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dock structures nearly 20 years ago, and he pinned his faith as much to it to-day as he had done then. Pre-cast work was undoubtedly cheap and efficient, due to the fact that it could be laid out very much better on a floor than on temporary structures over water. Inspection also was facilitated; it was easier to see that the steelwork was in its place and that the concrete was properly deposited. There was also the additional advantage that, when the work was completed and the members were matured, they constituted a permanent, instead of a temporary, support for any work, which had unavoidably to be carried out in situ. He had been accused of over-indulgence in reinforced concrete, but he claimed that he was, at any rate, sensible of and conversant with its limitations, one of which was the condition that the work must at all costs be designed so that as far as possible no damage could take place below water, particularly in view of the extraordinary rectangular midship section of modern ships. That, no doubt, was what the Author had had in mind when he designed the overhanging coping. It would be interesting to know, however, if there was any local or other justification for such an overhang as 5 feet. He imagined that 18 inches or 2 feet would have been sufficient, notwithstanding the fact that bilge-keels often projected almost beyond the beam of the vessel. He did not agree with the Author's recommendation of close timber fendering. Plain balk timber, so long as both vertical and horizontal fendering was provided, was certainly cheaper in first cost and much more efficient so far as maintenance was concerned. Thin board fendering on the front was an additional expense, and it continually got damaged, whereas with plain balk timber only that was not the case. He agreed with the Author as to the inefficiency of rope-moored floating plant for pile-driving, cylinder-setting, etc. It was much better to adopt shore plant, which, however, also had its drawbacks. He had got over the difficulty very efficiently in two or three cases by providing a barge which carried one end of a staging, the other end of which was fixed to a carriage on the shore. On this staging the whole of the pile-driving apparatus and the crane were erected. That plan had the advantage that the whole area of the wharf could be much more efficiently covered without the necessity of having any long crane-jib reaches, and the further advantage that the staging was always square with the work. What method had the Author adopted for sinking the cylinders, particularly the front ones? He had been rather surprised to find that the penetration was somewhat small, in view of future deepening or even allowance for varying depths when cleaning up the canal-bed. Dredging could not always be guaranteed to

Mr. Cleaver.

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Mr. Cleaver. a dead level, and in order to provide for that deepening, which often took place when cleaning out a berth, he liked to see the cylinders at least 2 or 3 feet below dock-bottom. He wondered whether the Author had had any difficulty in getting them down lower than the level shown on the plan. He could see nothing in the Paper to justify the adoption of the cruciform steel piles, in preference to reinforced-concrete piles. He was inclined to believe that, if the latter had been used throughout, they would have been cheaper than the steel piles; and, further, that unless there was extreme necessity for the exceptional loading, shorter piles, similar to those adopted under the sheds, would have been found ample in the front. It would be interesting to know if there had been some special reason for using steel piles in that particular instance. He agreed with the Author regarding the McKiernan-Terry hammers. He had found them to be very unreliable when used for permanent work. No doubt they were exceedingly useful for temporary and quick work. He had tried them on sheet piling and had driven the piles to absolute refusal. He had then followed on with an ordinary monkey, and in almost every instance he had been able to drive the piles several feet lower. He was astonished at some of the pile-driving results and sets which the Author gave. The strata at Manchester must be exceedingly elastic or soft, and very different from the strata to which he had been accustomed, namely, compact boulder clay and gravel; and the engineer who attempted to drive a blunt-headed or shoeless pile in that material would have an unpleasant surprise. He had never been able in South Wales to drive into gravel any piles with even a blunt shoe; he had had to make acute angled shoes specially in order to get the penetration desired. The sets mentioned by the Author—4 or 5 inches for 20 blows and upwards—seemed to be unsatisfactory. The results of the test load were interesting and valuable, but he did not think that with a set of that order full use was made of the sustaining power of the piles; a pile was not, in his opinion, properly driven unless the set was 1 inch, or less, in 20 blows. There was considerable danger in excavating for cylinders, or even in pile-driving or dredging for a wharf of the character under discussion, after the foundations for the shore sheds were completed. If it was at all possible, the wharf should be completed first, and then the foundations and the pile-driving for the sheds could be proceeded with quite safely. If that had been possible at Manchester, the movement which the Author mentioned would probably not have taken place. That was why he was surprised to find that the rear deck beams had been made in situ, instead of by pre-cast work, as in the latter case they would constitute

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very efficient struts. He agreed with the Author as to the superiority of steel forms. In the only instance in which he had been able to persuade a contractor to use steel forms the work had proved to be the cleanest and most satisfactory he had ever done, but the cost was high. The trouble was to get a contractor to face that cost, unless there was a considerable amount of repetition work. Gravity shoots were mentioned in the Paper. He would not use them on any account, for they were open to the very serious objection that the concrete had to be abnormally wet, and to his mind that was one of the weaknesses in the methods for carrying out reinforced-concrete work. He was inclined to think that a very considerable proportion of failures was due to the use of those gravity tubes. Separation, in his opinion, must take place. The concrete had to be unduly wet before it would travel down the shoot, and when it was wet enough to travel the distances which were expected with tubes of that kind, the heaviest particles naturally attained the highest velocity: the result was that when the concrete reached its destination, it was not in anything like the proportions intended when it was mixed. He was interested to note on p. 87 that the estimated pull necessary to extract piles which had been driven 20 feet into the ground was 100 tons. Some years ago he had to extract a pile which had been driven about 45 feet to refusal. In that case two 100-ton hydraulic jacks were needed even to start the pile, and one of the jacks was practically ruined in the attempt to carry out the work. That would give some idea of the difference between the ground at Manchester and the ground of which he himself had had most experience. Mr. Cleaver.

Mr. Alfred Hiley wished to confine his remarks to the subject of pile-driving. It was only on rare occasions that a Paper contained sufficient data to enable the pile-driving operations to be analysed, and the present Paper lent itself admirably to that purpose. The cruciform steel piles, driven by a McKiernan-Terry hammer, were intended, when they were finished in the cylinder, to take loads of about 90 tons. In opposition to an opinion expressed by a previous speaker, he thought the hammer had an appropriate application in that instance. A hammer of that type consisted of a heavy casing which surrounded the reciprocating piston, the latter being of small mass compared with the total weight of the hammer. The object of the casing was to preserve the necessary steam-pressure to give the light piston the high striking velocity of 20 feet per second. The hammer in question had a ram only 1,250 lbs. in weight, whereas the weight of the pile, including a steel anvil of 380 lbs., was four and a half times as much. Calculation showed that, having regard Mr. Hiley.

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Mr. Hiley. to the relative weights of the pile and the ram, an efficient blow was given, the striking energy in this instance amounting to the weight of the piston or ram multiplied by an equivalent free fall of 6 feet. That energy was sufficient to develop in the pile in question, taking into account the efficiency (45 per cent.) of the blow delivered and the work expended in elastic compression of the pile (which was small in steel piles), a force of about 100 tons. Therefore the hammer was capable of doing its work; but it would not have been suitable if the weight of the ram had not been sufficiently large compared with that of the pile. The concrete piles weighed more than 3 tons each; and if a drop hammer of 1,500 lbs. weight (ignoring the McKiernan-Terry hammer) had been used on such a heavy pile, it would not have been possible to overcome a heavy resistance. It was necessary to have a due proportion of energy in reserve for combating the elastic compression of the pile itself, and in the instances described in the Paper first a 2½-ton hammer and later a 3-ton hammer were used, because they gave a higher efficiency of blow and, in addition, greater energy. It was desirable, having ascertained the efficiency of the hammer-blow, to know the work expended in elastic compression of the pile. It was vital that engineers should thoroughly appreciate what was implied by the sets obtained, and he thought it was astonishing that that feature, dealing with the matter of producing genuinely sound foundations, was very rarely understood to a sufficient degree. He had developed and explained[1] a rational pile-driving formula, and had shown how the measurement of the driving force and the sets obtained could be interpreted to give useful information as to the resistance. The formulas which the Author had made use of were very poor guides —they were certainly outclassed at the present day. The Wellington formula had been introduced in America in connection with the driving of timber piles; it did not take into account the relationship between the weight of the ram and the weight of the pile. For instance, that formula could be applied to a pile 2 feet long, or to a pile of the same section 40 feet long, and it would indicate the same driving force from a measurement of the set obtained per blow. That could not be so, because the efficiency of the blow had been entirely neglected. The constant which the Author modified was, after all, only a figure introduced in an attempt to bring the result nearer to what could be ascertained to be the safe load; it also referred to the temporary compression of the pile, but it did not do so in a scientific manner. The Dutch formula took

[1] *Engineering*, vol. cxiii (1922), p. 673; and *Engineering* vol. cxix (1925), p. 657.

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no account of the temporary compression of the pile, which to a very inadequate extent was dealt with by its companion formula; but it took into account the efficiency of the blow, estimating that efficiency as derived from driving a pile with a ram striking an inelastic surface. It allowed nothing for the temporary set which occurred due to the elastic compression of the pile during driving, and therefore indicated exaggerated resistances, calculated for sets approaching zero. If the Author had referred to the work which had been carried out by Mr. Ackermann and Mr. Hiley in their investigations on pile-sets and allied data[1], he would have found much that would have saved expenditure on the driving of test piles, and in other directions would have indicated where confidence was warranted. The rational pile-driving formula which Mr. Hiley had already referred to, and which he intended to use for the purpose of criticizing several speculations contained in the Paper, would enable a broader view to be taken of the essential facts recorded, and consistent relationships to be established. It involved the following elements:— Mr. Hiley.

The striking energy of the ram . .	wh
The efficiency of the hammer-blow . .	η
The set per blow	s
The temporary compression of the pile .	c

The resistance overcome, $R = \frac{wh}{s + c/2}$ multiplied by η

This formula enabled the driving resistance to be computed with equal facility for all classes of hammers and piles, and when applied in the present instance allowed the useful facts disclosed in the Paper to become better understood. *Fig. 15* showed the curves of driving resistance and safe loads, corresponding with the Author's *Fig. 11*; the curves marked (1) and (2) were taken from the Paper to demonstrate the misapprehension entailed in accepting the fictitious factor of safety of 6 involved with the *Engineering News* formula and the factor of 5 with the Dutch formula. The full-line curves showed the safe loads on the pile and the curve for 75 per cent. of the driving force indicated the extreme limit of loading for a pile, beyond which a commencement of settling would give occasion for concern. The point B showed the relation of the maximum test load to the danger limit, and the point A confirmed the safe condition as found for the actual test load of 60 tons, for which no settlement beyond the natural compression of $\frac{3}{16}$ inch for the loaded 35-foot

[1] Trans. Soc. Engineers, 1923, p. 125. "The Impact of Imperfectly Elastic Bodies with particular reference to the Effect of the Hammer Blow in Pile Driving."

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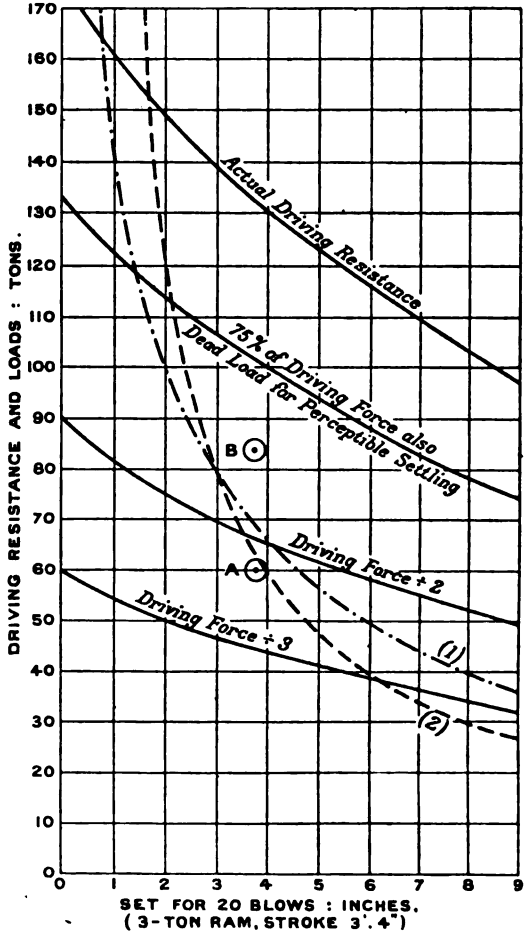
Mr. Hiley. piles was experienced. The maximum safe load for a pile should not, in practice, be permitted to exceed one-half of the driving resistance, if safety against settling had to be ensured. That the Author had succeeded in keeping his loadings in the main below the limit where untoward results would become apparent was a matter for

Fig. 15.



congratulation, although it could not be conceded that this result had been secured through adequate scientific control of the problem. In view of the policy he had followed the Author had manifestly taken a wise course in testing one cluster of piles, and had thereby derived a valuable check on what were otherwise wild hazards

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concerning the safe loads, as indicated by the American and Dutch formulas. *Fig. 15* should convince engineers that the position of affairs in regard to pile-driving resistances for permissible working-loads could not be allowed to rest where it was, especially considering the fact that this field had been explored during the last few years, and reliable guidance was now available. On analysing the facts brought to light in the Paper, the following summary of interesting comparisons would be found. (1) The 61-foot cruciform steel piles (p. 70) had been driven to develop a resistance of 90 to 100 tons, and the No. 9 double-acting McKiernan-Terry hammer showed an efficiency of 45 per cent., and was therefore appropriate for the case, for which the ratio of the weight of the steel pile to the 1,250-lb. ram was 4·3. (2) The 14-inch by 14-inch reinforced-concrete piles driven with a 3-ton ram having a drop of 3 feet 4 inches (p. 72) developed, on reaching rock, a resistance of 150 tons. The pointed-end resistance would not be less than 50 tons, the remainder being taken by side friction. As the area of concrete immediately above the cast-iron point was only 36 square inches, the stress in the pointed end exceeded the strength of the 1 : 1½ : 3 concrete—approximately 2,000 lbs. per square inch—and consequently the points had, in all probability, partially crumpled up, although the Author expressed the view that the rock had been penetrated to the depth of about 1 foot. (3) The trial piles driven with a 2½-ton semi-automatic hammer, using a 3-foot stroke, and striking on a 4-foot dolly (p. 75), would develop 140 tons resistance at $\frac{1}{6}$ inch set per blow. The same piles, when redriven by a 3-ton semi-automatic hammer, using a 2½-foot stroke (p. 79), indicated an initial resistance of 175 tons, which again diminished to 140 tons when the set per blow became $\frac{1}{20}$ inch. The augmented resistance revealed after a lapse of 12 months, for the class of strata encountered, amounted on the average to 25 per cent. in excess of the driving resistance, owing to the uplifting action disturbing the compactness of the ground when the pile sprang back to its true length after being temporarily compressed during driving. The redriving data (pp. 79 and 85) furnished good records and corroborated this natural behaviour. In the instances of redriving after the 12 months, the first resistance of 130 tons became 155 tons, an increase of 20 per cent., on resumption of driving. It was quite a mistake to attribute the effect to cushioning by the dolly. (4) Mr. Hiley would warn the Author against pinning his faith to his amended formula, seeing that for some new set of conditions its use was more likely than not to obscure the true facts of the case. (5) The contrast found in the results of driving given on p. 78

Mr. Hiley.

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Mr. Hiley. for the No. 9 B double-acting hammer, which had a kinetic member of merely 1,500 lbs., and for the more efficient single-acting hammer with a 3-ton ram, appeared to have caused surprise. To facilitate the intelligent selection of the hammer for such a case, Mr. Hiley had set out[1] the relevant data pertaining to the energy given out by the various sizes of hammers, and had explained how far their serviceability extended. Those particulars showed that the McKiernan-Terry type reached the limit of its service in driving heavy piles to a resistance of 80 tons where the set was $\frac{1}{10}$ inch per blow. Likewise the single-acting hammer, producing a set of 0·6 inch per blow, would be found to have also met with a resistance of 80 to 85 tons, without any anomaly entering into the comparison. The method adopted of steadying the single-acting hammer by suspending it from a crane was unusual, and, he believed, entailed a problem in designing the guides so that no sudden accelerating force would act on their mass at the instant of impact. He would appreciate details of the device used, if the Author could see his way to explain it. With regard to the 11½-inch diameter elm buffer used to transmit the blow evenly on to the pile-head of 100 square inches only, with a driving resistance of 130 to 150 tons the smallness of the area provided—would appear to intensify the stress in the pile-top, and in the wood itself, to the limit of endurance in many instances for both the parts. It would be interesting to learn the extent to which pile-heads had suffered damage when the sets were less than 3 inches per 20 blows of full stroke. Presumably in some cases the choice of 3 feet 4 inches for the stroke (the full stroke available being 4 feet 6 inches) had been influenced by experience with the first set of test piles, and he asked to what extent this might have been the case. The Author's experiments to determine what effect different degrees of pointedness of the piles had on the total driving resistance showed results quite in harmony with the nature of the case. Considering the point of a pile to be that portion below the level where the parallel section finished, Mr. Hiley had ascertained from experiment that, for points which were completely embedded in the ground, a half angle of 20 degrees showed the least resistance, which was 66 per cent. of the value given by a flat-ended pile, reckoned apart from the frictional resistance of the sides. Half-angles of point of 45 degrees gave a resistance equal to 75 per cent., and of 15 degrees gave 70 per cent. of that due to the corresponding flat end. Thus 20 degrees, as generally adopted in practice, was well chosen. A shoe of this angle

[1] British Steel Piling Company's Pocket Book.

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was less liable to become crumpled up when meeting boulders or rock, as might easily occur with the delicate design of over-sharpened 11 degrees half-angle of point made use of at Trafford wharf. If sharper points than 20 degrees half-angle possessed any merit at all, which was more than doubtful, it could only be claimed that in the early stage of driving through the crust of the ground their directional qualities when pitching the pile would better serve to keep it in its desired position. If he had criticized the matters relating to the piling described in the Paper a little acutely, in order to emphasize the lessons which the Paper conveyed in such a typical case of present-day piling practice, he did not wish to imply any lack of appreciation of the painstaking endeavour which the Author and his colleagues had devoted towards making their work sub stantially successful. In that respect he believed the work of reconstruction at Trafford wharf would long remain prominent as having contributed in no small measure to bring about a realization of what could be done to bring the art of piling out of its state of darkness into the light of science, to the permanent advantage of engineering. Mr. Hiley.

The AUTHOR, in reply, remarked that his experience of blocks in connection with moles and breakwaters, had been that the unit costs of large and of small blocks came out to practically the same. Small blocks, say of 5 tons, handled by a small crane, were no more easily dealt with than were 30- or 40-ton blocks put in by a big crane. In the case under discussion he thought the big block made the better job, as one of the considerations had been the question of ships coming alongside the wharf. He had made the blocks heavy so that the ships should not shake the structure out of position. The Manchester Ship Canal Company had two floating cranes, and it had been agreed that the contractors should have the block-yard elsewhere, load the blocks on to barges, bring them up, and unload them from the barges and put them into the work with the Company's machines. The contract had been let on those lines, but the contractors had elected to make the blocks on the site. The blocks had been in such a position that the floating cranes could not get at them, and the contractors therefore had had to use shore tackle, the cost of which was only £3,000. That machine, therefore, was not expensive, and it was well worth its cost; incidentally it had released the Company's machines for their own work. With regard to the bonding of the wall-boxes together, each block had a 4-foot diameter semi-circular hole at the end to its full height, and mass concrete had been put in to make an efficient key between each pair of blocks. The subway when he last saw it was in a thoroughly good condition, there being no cracking. The asphalt from end to The Author.

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The Author. end was good and was dry. Asphalt had been used in all the sheds in Manchester; the only matter calling for particular care in putting down asphalt floors was that the concrete should be quite level underneath, as otherwise trucking would in time cause the irregularities in the concrete to be repeated on the surface of the asphalt. The overhang of the front of Trafford wharf was more than would be used in the ordinary way; but it was necessitated by the design of the cylinders, which were under the front boxes in the best position to carry the superimposed loads. He had just put in piling for two other sheds, and in that work he had tried a 4-foot dolly to see whether it made any difference in the driving of the piles, and whether he could measure such a difference in any way to make a formula. Although there was a slight slowing in the rate of driving the pile, the difference could not be represented by a formula. He had stated (p. 78), that there did not appear to be any reliable formula for the McKiernan-Terry pile-engine, and he agreed with Mr. Wilson on that point. He had modified the Wellington formula for this work because the test loading showed that the group of piles driven to 3·8 inches set for 20 blows sustained a load of 60 tons without undue settlement. The Wellington formula gave a safe load of 69 tons for this set; he had modified the formula to bring the load on the ground encountered within safe limits, and had proportioned the number of piles in the various foundations to suit the modified formula according to the sets ascertained by driving a number of test piles over the whole area of the foundations. The reinforced concrete in the boxes and decking had been made richer than the mass concrete, owing to the complicated steel reinforcement, which required careful covering and surrounding with concrete of good quality. The question, asked by Mr. Roberts, as to the depth to which the reinforced-concrete piles were driven in the rock was answered on p. 72; the piles near the bore-hole, sunk specially to ascertain this information, went about 1 foot into the rock, or to about two-thirds of the depth of the cast-iron shoe. The Author's experience with regard to timber fendering was that with open balk fendering the ship's fenders and plating were continually damaging the timberwork, and continuous fendering was found to be less costly in maintenance. The fendering was not "thin boards" in the case under discussion; it was 6 inches thick—for dolphins it was 12 inches thick—and for a wharf now to be constructed where there was a considerable rise and fall of tide, and where vessels would pass at considerable speed and cause ranging of those moored alongside, the continuous fendering was to be 9 inches thick. The cylinders had not been sunk as suggested by Mr. Cleaver, but had

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been placed in position over the piles after they had been driven, the ground having been excavated to the required depth; the primary function of the cylinder was to protect the steel piling from deterioration and give it additional stiffness. The Ship-Canal had already been deepened and the Trafford wharf berths dredged to the level of the lock-sills before the construction of the wharf was commenced, and therefore until additional locks were constructed no further deepening would take place. If additional depth was then found necessary, the 1½-to-1 slope could be extended to the increased depth without endangering the front cylinders, which were founded on good hard clay. It had been decided to use steel piles in preference to reinforced-concrete because fewer were required to sustain the loads, and therefore the cylinders containing them could be smaller and less costly than for concrete piles as first contemplated; a further reason was that the Author considered it would be wise in any case to drive the piles for the two rows of boxes to rock, and long piles (up to 70 feet) were required, which would be more easily driven if of steel. It had originally been proposed to pre-cast the rear deck beams, but the contractor elected to cast them in situ; this was agreed to, and advantage was taken of this mode of construction to put in additional steel to tie the back line of concrete boxes to the shed foundation-blocks; the use of the decking as a strut, in case of heavy blows by ships, was improved rather than otherwise by this method of construction. Mr. Cleaver's case of two 100-ton jacks being used to draw a pile 45 feet long was not dissimilar to the experience at Trafford wharf, where the pile 20 feet in the ground had required a 100-ton pull to withdraw it. The formula used by Mr. Hiley was interesting, and his observations on the Paper were very instructive; but his factors for the efficiency of the hammer-blow and the temporary compression of the pile could only be adopted with great caution, as improper use might lead to results which would not work out satisfactorily in practice. The Wellington formula was really comparable with Mr. Hiley's formula, factors having been adopted in the Wellington formula which had been found to give practical working results. Taking the formula on p. 99, $R = Wh\eta\,/[S + (c/2)]$, if the value of η were taken as 0·5, c as 0·20, and the factor as 3—which were practical values for working within normal limits—the result was the same as would be arrived at by using the Wellington formula. For instance, using the following values in the formula given above:—$W = 3$ tons, $h =$ 40 inches (3 feet 4 inches), $S = \frac{1}{4}$ inch (5 inches for 20 blows), and $c = 0{\cdot}20$, $\eta = 0{\cdot}50$, the value of R was found to be 171⅔ tons, and $R/3 = 57\frac{1}{7}$ tons. By the Wellington formula P also equalled 57⅐ tons

The Author

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The Author. for the same value of S. With the modified Wellington formula adopted at Trafford wharf, using 0·15 in the denominator (which compared with $c = 0{\cdot}30$ in Mr. Hiley's formula), and taking the other values as above, the values of $R/3$ and P both worked out at 50 tons for 5 inches set for 20 blows. It was understood that in making the calculations for the curve of "actual driving resistance" on *Fig. 15*, Mr. Hiley adopted a value of 0·53 for η, but took variable values for c and for the drop h. The drop of 3 feet 4 inches had been adopted because it was a reasonable one, easy to use in calculation, and less than the full stroke of the hammer, so that it was always certain that the full effective drop was being obtained. Mr. Hiley stated that "the maximum safe load for a pile should not, in practice, be permitted to exceed one-half of the driving-resistance"; that line of permissible loading was shown on *Fig. 15*. It appeared to the Author that the loads thus given were not very satisfactory; they were too high for high sets—for instance, with a set of 9 inches for 20 blows the permissible load was 50 tons, against 33⅓ tons according to the Author's calculations—and for low sets they were too low. For instance, for a set of 1 inch for 20 blows the permissible load was given as 82 tons, and for refusal 90 tons, the Author's values being 100 tons and 133 tons respectively (*Fig 16*, curve (3)). For conditions of driving similar to those at Trafford wharf the Author would prefer to use the curve (3), *Fig. 16* (which was the Author's curve on *Fig. 11*), as being more conservative for high sets and necessitating fewer piles for low sets. Of course, with low sets the load that a pile would safely sustain as a column would have to be carefully considered. With regard to the penetration of the pile-shoe into the rock to a depth of about a foot, there was no reason to believe that that was not correct; the surface of the New Red Sandstone rock was usually softer than the rock below, and the steel rods in the piles were brought together at the bottom and were in contact with the top of the shoe, thus transmitting the blow of penetration direct; under such conditions the concrete in the pile close to the shoe had probably not suffered any damage. The pile-heads had been somewhat damaged when the sets were high, the concrete spalling off for about a foot, and the steelwork being exposed; in some cases the steel rods had been slightly bent for a few inches down. Nothing had been done afterwards except to clear away the loose concrete where the heads were buried in the mass concrete foundations. The method of steadying the single-acting hammer by suspension from a crane was as follows. Two octagonal straps were bolted around the head of the concrete pile, and to these were welded two channel guides, fitted back to back with a space

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between, and long enough to clear the hammer and piston gear. The guides were suspended from the crane at the top; the two lugs on the hammer passed through the space between the guides and

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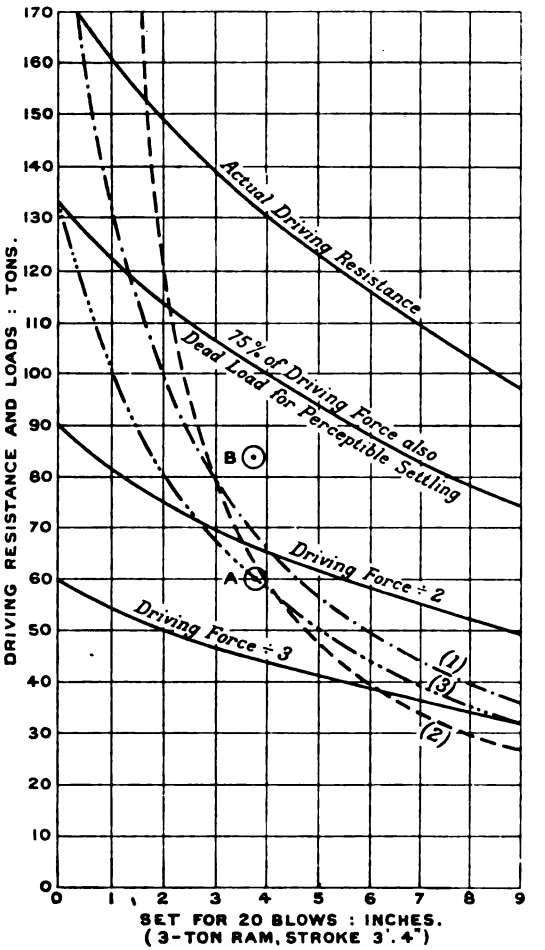
Fig. 16.



were retained in place by rollers at the back, so that the hammer when moving up and down was controlled and guided by the channel bars. The crane rope was lightly held on to the brake of the crane and was eased off by the crane-driver as the pile descended.

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Correspondence.

Mr. Ackermann.

Mr. A. S. E. ACKERMANN considered that a pleasing feature of the Paper was that it contained so many details, thus enabling deductions to be made from the results given. The Author seemed surprised at the effect of the 4-foot dolly used in connection with the test piles. The following experiments with three 3-inch wire nails, however, would easily prove that the shorter the dollies were, the better. One of the three nails should be driven into a solid piece of timber with a hand-hammer in the ordinary way. In the case of the second, a ½-inch slab of indiarubber should be placed on top of a strip of ⅛-inch iron plate, a wooden lath being placed on top of the rubber in order to hold it in position. If this combination were placed on top of the nail so that the rubber was centrally over it, an attempt to drive the nail by hitting on the lath would show that the difference produced by the cushion on the effect of the blows was very marked. The effect of inertia, which came in especially in the case of heavy piles, and those with steam pile-drivers attached to them, could be illustrated by interposing a heavy piece of iron between the head of the nail and the hammer; again it would be found that the blows of the hammer were far less effective in driving the nail than when the nail was hit directly.

He had attached his autographic pile-set gauge to a pitch-pine test pile, 15½ inches square by 33 feet long, and had found that a 20-foot length of this pile was elastically compressed as much as ¼ inch, due to a hand-winch-operated hammer, weighing 1 ton, falling through a height of 4 feet 3 inches. When a hammer was used for driving a pile, its energy might be used in one or other of two extreme ways. Either it could cause the pile to be compressed and move only a small distance, in which case the maximum force of the blow would be great; or the compression and movement might be considerable, in which case the maximum force of the blow would be much less. The diagram of the work done on a pile, up to the point of the maximum force being attained, was a triangle, for the maximum resistance of the pile did not occur until it had been compressed to the maximum extent. The areas of the two triangles in the extreme cases, or the areas of the triangles representing any intermediate case, were of necessity all equal; hence anything which increased the amount of elastic compression decreased in direct proportion the maximum compressive force due to the blow.

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Mr. Ackermann.

The Author stated that there appeared to be no reliable formula to give the safe load for a pile driven by a McKiernan-Terry type of steam-hammer. Mr. Ackermann was of the opinion that there was, at present, no reliable formula in connection with any form of percussive pile-driving, with the possible exception of the method of dealing with these problems derived by Mr. Alfred Hiley, who used the values of the moduli of elasticity of the different materials involved in order to obtain his results. Mr. Ackermann doubted (owing to the many factors which had to be taken into account, and because the moduli of elasticity of the particular materials had to be assumed) whether even the results so obtained were entirely reliable, but he agreed that they were far more likely to be so than the results given by other formulas. Mr. Hiley's principle could be illustrated by means of Mr. Ackermann's results obtained in connection with the compression of the pitch-pine pile already referred to. In his fourth Paper[1] on "The Physical Properties of Clay," he had shown how the modulus of elasticity of the material of the pile could be calculated when the force of the blow and the elastic compression were known. Conversely, if the elasticity were known, the force of the blow might be calculated in the manner shown by Mr. Hiley; but Mr. Ackermann considered it was more satisfactory to attach his autographic pile-set gauge to any particular test-pile, and by means of it to measure not only the elastic and permanent sets, but also the duration of the blow, and consequently the maximum compressive force due to the blow. Possibly this was the only trustworthy method of determining the actual forces involved in pile-driving. To show how thoroughly unreliable the existing formulas were, Mr. Henry Adams, M. Inst. C.E., had applied thirty-nine different pile-driving formulas[2] to the data referring to one particular pile, with the result that he obtained thirty-nine widely different results! The "ultimate loads" ranged from 37 tons (Hurtzig) to 333 tons (Rankine), and the "safe loads" from 5 tons (Rivington), to 67 tons (Rankine).

The potential energy of a pile-driving hammer was expended in several ways:—

(1) Overcoming the friction of the leaders.
(2) Causing damage ("brooming") to the head of the pile.
(3) Producing heat in the head of the pile.
(4) Elastically compressing the dolly and pile.

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[1] Trans. Soc. Engineers, 1922, p. 170.

[2] "Pile-driving and the supporting Power of Piles." Trans. Concrete Inst., vol. viii (1916).

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Mr. Ackermann.

(5) Overcoming the inertia of the pile, and of the attached steam pile-driver if used.

(6) Overcoming the friction of the soil on the sides of the pile.

(7) Overcoming the vertical reaction of the soil on the toe of the pile.

(8) Elastically compressing the soil under the toe of the pile and stretching the clay adhering to the sides of the pile.

In the case of a particular test pile (a "Simplex" steel tube, 30 feet long, 16 inches in external diameter and 2·1 tons in weight, driven by a 38½-cwt. hammer, dropped in all cases about 5 feet) to which his autographic pile-set gauge was attached, Mr. Ackermann had been able to determine the more important of the foregoing eight quantities, with the following results:—

	Inch-tons.	Per Cent.
Energy necessary to overcome the inertia of the pile .	25·7	22·3
Energy lost in elastically compressing the pile and soil	38·6	33·4
Energy available for overcoming the resistance of the soil to the penetration of the pile	51·2	44·3
Gross potential energy of the hammer . .	115·5	100·0

The dolly used was about 10 inches long, being of hard wood inserted in a steel container, which latter rested on the top of the pile tube.

He considered it was a pity that when the redriving of the test piles at Trafford wharf was decided upon, that was not first attempted with the use of the 4-foot dolly, so as to reproduce, as the Author stated, "the former conditions as nearly as possible." After the results of using the dolly had been recorded, it might have been discarded. He agreed that the difference was probably due to the use of the 4-foot dolly; consequently the moral was, use as short a dolly as possible. The comparison of the pointed and blunt piles, given by the Author, confirmed Mr. Ackermann's results with model piles, which were recorded in his five Papers on "The Physical Properties of Clay," read before the Society of Engineers during the period 1919–1923. The Author's explanation why the blunt pile offered less resistance than the pointed one was that given in those Papers. This result had been obtained by Mr. Ackermann, even though his piles had been forced in gradually by periodically increasing the loads, and not by percussive blows. He would expect the difference to be more marked in the case of piles driven by hammer-blows, because with the square-ended pile, the clay would squirt sideways, at right angles to the pile, more than in the case of a slower motion, and thus would be in less intimate contact with the sides of the pile.

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Mr. Ackermann.

One engineer had stated that with flat-ended piles, the clay freed itself entirely from the sides of the pile. Mr. Ackermann had made similar experiments, but had never found that there was no contact between the clay and the sides of the pile, or anything approaching to that condition. Not only had he investigated that point by probing in a suitable manner between the pile and the clay, but the results, when plotted with those of other experiments, in which side friction had been entirely eliminated, clearly showed that there was plenty of contact between the sides of the pile and the clay. That merely confirmed, what was well known to those who used piles, that in the case of piles which were not resting on a hard stratum, the friction on the sides of a pile supported more of the load on the pile than did the vertical reaction of the clay on the toe of the pile. In fact, he had shown that in one case about four-fifths of the ultimate load on the pile was supported by friction.

Mr. Duckham.

Mr. F. W. DUCKHAM considered that the Paper formed a very complete and acceptable record of a particularly interesting work. The design of the reinforced-concrete work, and especially that in the large girder boxes, was of such a novel and enterprising character, that he felt it a special privilege for his firm to have been instrumental in carrying it into effect. This novelty in design had had, in turn, the effect of stimulating the contractors to similar enterprise in improved methods of construction, such as the Author indicated, in the moulding and setting of the large box castings, the driving of the piles in groups without the use of pile-frames, and other such procedure, which had entailed the Author's approval before commencement. The general result showed an economy both in time and money, which had been expressed to the mutual satisfaction of clients and contractors.

Mr. Halcrow.

Mr. W. T. HALCROW considered the Paper to be of great value to engineers responsible for designing heavy piled foundations. The comparative results of using a steam hammer of the McKiernan-Terry or similar type and an ordinary drop-hammer, were instructive and confirmed his experience that bearing piles should be driven with the latter type, although the former was probably the more efficient for sheet piling. The redriving tests were a useful record. He had carried out some similar tests on a reinforced-concrete pier constructed by his firm 20 years ago. Piles 14 inches square, of an average length of 50 feet, were driven 18 feet into a sand foundation, the final set being obtained with only one-quarter to one-third of the number of blows specified. In several cases the piles were retested after intervals of 1 to 3 months, and the results

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Mr. Halcrow. obtained were two-and-a-half times the number of blows previously required. Although the results were then still below the specification, no additional piles were driven, and there was no evidence of any settlement. Where piles were required to carry their load by frictional resistance, the square-sectioned pile had certain advantages. Comparing the 15-inch octagonal with the 14-inch square pile, the cross section of the latter was about 5 per cent. greater, but the surface area was 13 per cent. greater and the weight of steel per foot slightly less. Group driving was shown to increase considerably the average set of the piles in the group. Reinforced-concrete piles 14 inches square in a pier now under construction had been driven in groups of three within a 4-foot 4-inch circle; and average results obtained were, for the first piles driven, 4½, for the second 5½, and for the third 6 blows to the last inch, by a 2½-ton hammer falling 4 feet, the piles being about 25 feet in sand and gravel. There did not appear to be any advantage in driving a group of piles in the manner described by the Author, namely, partly driving one pile and then moving to another. Mr. Halcrow's experience was that as good average results were obtained by driving each pile to its full depth.

Mr. Hollingworth. Mr. E. W. HOLLINGWORTH remarked that the maximum safe load for a pile was very difficult to determine, and the full particulars given in the Paper were correspondingly valuable. The varying results showed the unreliable nature of pile-driving formulas. The fact was that the set of a pile depended almost as much on its own condition as on the resistance of the ground; and a timber pile with a soft or spongy head would appear to meet with more resistance than a pile which was hard and sound throughout. The cushioning effect of dollies and caps could only be guessed, and it increased as the timber broke down under repeated blows, so that a formula could only be rendered trustworthy by the adoption of caps of some soft metal or other material much more uniform than elm timber.

Mr. Latham. Mr. ERNEST LATHAM regarded the Paper as courageous, and the frank way in which the difficulties of constructing the wharf were dealt with was refreshing. The Author admitted that the design was unusual. Undoubtedly that was so, but there were hardly ever any two instances in maritime work where the same conditions prevailed; and Mr. Latham would, therefore, refrain from dealing with minor details. As he understood the Paper, the cylinders had been built up of annular sections in reinforced concrete and ultimately filled with mass concrete. He agreed that the use of steel moulds was good practice, and he had recently been, and was at present, utilizing them in constructing cylinders in the estuary of the Thames in 30 feet of water at low

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tide. The essential difference in the latter construction, however, was that the cylinders were cast in a single mould in situ, a short section of mould being left in at the bottom, and the whole interior volume between the levels of the foundations in the river-bed and low-water level was poured in one tide. Remarkably good results had been obtained by this method, which was advocated by the contractors concerned. An excellent feature in the Author's design was the close horizontal fendering adopted, which precluded projections from the sides of ships from fouling either vertical or horizontal isolated timber members. A point open to criticism was the selection of the pre-cast method of constructing the superstructure, in preference to constructing the work in situ. The water-level appeared to be substantially below the superstructure, but the water-level in the dock would presumably vary slightly from time to time, and it would be interesting to know what the maximum level was. The greatest value of the Paper, however, lay in the length at which the Author had dealt with questions of pile-driving, and probably no other Paper of its kind contained so much valuable data. The sets of the cruciform steel piles of the wharf proper were, of course, beyond all question; but, in spite of the test loads put on the reinforced-concrete piles, the sets of the piles for the transit-shed foundations seemed to be poor. The Paper again raised the question of standardization of terms and formulas in connection with pile-driving. The concrete piles apparently penetrated more per blow than did the trial piles at the finish of driving, and that was expressed by the Author as "the actual sets obtained were actually much in excess of those obtained at the trial." With all respect to the Author, he thought the average engineer, if he did not refer back to the results obtained on the test piles, would regard this as expressing stiffer driving. There was a good deal of confusion in the use of the terms "set" and "penetration," and there was still much to be desired in connection with standardizing formulas which could be relied upon to give reasonable figures for the safe loads which piles could bear. He thought a good deal of the experimental work of Mr. Ackermann on models might be misleading in dealing with actual work on a larger scale. The Dutch formula, and that given by the British Steel Piling Company in its pocket-book as suitable for single-acting steam-hammers, both introduced a new factor, namely, the weight of the pile; while in Nichols's (U.S.A.) formula for the McKiernan-Terry hammer that factor was omitted. Theoretically, the inertia of a pile and its

Mr Latham.

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Mr. Latham. coefficients of elasticity, etc., did enter into the calculation of what safe load the pile could bear; but it must be borne in mind (and it would be well appreciated by those who had frequently watched heavy pile-driving) that the penetration of a pile was practically instantaneous as the hammer hit its head; and it was probable that nearly the full force of the blow was transmitted to the pile-shoe and the submerged faces of the pile at each blow, while the proportionate amount of energy transmitted did not vary very much as between timber, steel, and reinforced-concrete piles. Proof of this, however, could only be obtained as the result of experiments on a full scale or by collection of data from many sources. He thought that the study of this subject should command the attention of the British Engineering Standards Association, with a view to the standardization of both terms and formulas. The problem was of world-wide importance and applied to structures of all kinds carried on piling. His own view was that the ultimate result of such an investigation would probably eliminate the weight-of-pile factor and the different coefficients. He thought it was evident the Author also took this view, and there was possibly some physical similarity in pile-driving to the ability of a relatively soft solid to penetrate a harder one, provided the velocity of the former was sufficient. The driving of timber piles into hard chalk suggested this as the basic idea.

Mr. Mitchell. Mr. JAMES MITCHELL remarked, with regard to the piles and the pile-driving, that it was not very clear whether the clay referred to was alluvial clay or boulder clay; there was in general a marked difference between the two from a pile-driving point of view. Boulder-clay, despite its name, contained in many cases only a very small proportion of clay, and it might or might not contain a large quantity of boulders. With reference to Experiment B (pp. 82–83), although the flat-nosed pile gave a greater terminal set than the sharper one, it required about 100 more blows to drive it down to the same level. This afforded interesting food for reflection as to the relative bearing-capacity of those piles, and as to the formula relied on in calculating it. The same questions were raised by the remarks (p. 84) that "on the whole, group driving gave better sets than single driving," and that "the order of driving or the position of the piles in a group did not appear to be a determining factor in the set." The Author drew attention to the great variation in the pile-sets, under what appeared to be similar circumstances. It was instructive, in watching the driving of a number of piles from start to finish, to observe the irregularities of set which often occurred, not only as between pile and pile, but during the progress

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of a particular pile—irregularities which, in many cases, could be explained only by the presence in the ground of obstacles such as stones. An experienced piling-foreman would coax a difficult pile down by continually varying the drop of the hammer—shortening it when the driving was hard, and lengthening it with easy driving. In similar circumstances an inexperienced man would damage the pile, and perhaps fail to get it down. Short of stopping a pile altogether, a stone might deflect or bend it, or might be carried down by it through a distance depending on the size and shape of the stone, the strength of the pile-point, and the resistance of the surrounding ground. A smooth boulder of, say, hard basalt, embedded in stiff ground, might lie so that the point of the pile-shoe just cleared it, and with its side next to the pile having the same slope as the side of the shoe. In such circumstances there was likely to be severe bending of the pile, and its resulting general direction would depend on the amount of the bending, and on the nature of the ground previously passed through. One of the evil effects of bending or altering the direction of the pile was that the blows of the hammer were apt to be non-axial, with resulting damage to the pile. Sometimes that could be partially or entirely obviated by altering the position or the inclination of the piling-frame, but often this was impracticable. It had to be kept in view that an amount of bending or of vibration (such as was produced by non-axial driving) that would be harmless in the case of a timber pile might cause serious cracking in a concrete one, especially where the pile was driven to the extremely small set sometimes called for. With reference to the 22-degree pile-shoe, while it did not appear to have any advantage over the more usual 32-degree one, the connection between the point and the straps of it was likely to be much more severely stressed in stony ground. It was mentioned on p. 87 that three of the piles had to be drawn after partial driving, owing to cracking. Had any further cases of cracking been observed? There seemed to be good reason for believing that many piles, especially concrete piles, were driven by hammers that were too light, and that much of the damage done to the piles was due to this cause. There did not seem to be a due appreciation of the importance of a proper relation between the weight of the hammer and that of the pile. Experiments made on the driving of a wooden peg, say, 3 feet by 2 inches by 2 inches, into stiff ground, by means of hammers of various weights, would show very clearly the effect of the use of too light a hammer in damaging the peg. The Author's remarks (p. 79) on the "very large cushioning effect" of the dolly used in driving the trial piles suggested the question whether it would not be better

Mr. Mitchell.

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Mr. Mitchell. to do away with all forms of cushion and drive direct on the pile. For this purpose the top end of the pile, for a length of, say, two diameters, might be strengthened by making the helical wrapping of the reinforcement there to a reduced pitch, putting more cement in that portion of the concrete, and, if necessary, adding a few short longitudinal steel rods. Experience of direct driving showed that, by using a hammer of ample weight, with a correspondingly short drop, concrete piles could be safely driven to a set sufficient for all practical purposes. Piles had sometimes, as at the front of a wharf, to be driven to a certain level, even when a sufficiently small set was obtainable at a higher level, but excessive hammering was not the best method of dealing with such cases. It was well to remember that there was ample evidence that, although the head of a pile might go down $\frac{1}{64}$ inch, it did not necessarily follow that the point had gone down. When, owing to the head of the pile being below the bottom of the leader of the piling-frame, a dolly was necessary, it would probably be advisable in all cases, but especially with concrete piles, to use one made of steel, so as to avoid the great loss of striking-force, and the waste of time occupied in repairing, which were involved by the use of timber. It was stated (p. 72) that the piles were driven by means of a "semi-automatic steam-hammer with a 3-ton monkey," but as the hammer was described farther on as a "3-ton semi-automatic hammer," it was not clear whether 3 tons was the gross weight of the hammer, or the weight of the striking portion. Attention might here be directed to the different uses to which the word "monkey" was put in reference to pile-driving. Sometimes it was used to denote the hammer, sometimes the lifting-gear, and sometimes even the piling-machine. It was, properly, the excellent descriptive name applied by workmen to the lifting and tripping device used with the simple drop-hammer, the idea being that of a monkey slipping down a pole, grabbing the hammer, climbing with it to the proper height and then letting it go. Referring to the decision (p. 81) to restrict the length of the piles to 40 feet, so that there might be no irregularity of settlement as between the piles previously driven and the others, it would appear that by increasing the number of piles in each foundation the same result was obtained as would have been effected by the lengthening of the piles.

Mr. Reinhold. Mr. G. C. REINHOLD, describing the methods adopted by the contractors, observed that the procedure in placing the concrete cylinders forming the supports of the two rows of pre-cast boxes had been to excavate the canal-bed and drive the cruciform piles. To compensate for any inaccuracy in the positions of the piles, steel skids

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were fixed to the face of the piles by a diver, and, these having been adjusted to the true position of the concrete cylinder, the latter was then threaded over the piles and lowered into position. The cylinder was lowered and set by a diver working around the cutting edge with an hydraulic jet. The decision to make the reinforced-concrete boxes on the site, and not at Barton as originally contemplated, was arrived at after consideration of the risks involved in the transport of such heavy units along the canal. Moreover, the handling was dependent upon the floating crane of the Canal Company, which might be otherwise engaged when required for that service, resulting in disorganization and delay to the work. A further reason was the fact that it was possible to cast the boxes from the travelling concreting-tower on the site of the works and obviate duplication of this plant for the reinforced-concrete boxes. Care had to be taken in the preparation of the platform upon which the boxes were built. The platform consisted of heavy timber balks laid upon the first two rows of shed-foundations, upon which the platform was built. The outer row of boxes had steel forms for both inside and outside faces. The inner row of boxes had wooden forms for the inside face and steel forms for the outside face. Owing to the large amount of reinforcement the concrete had to be very thoroughly rammed, to ensure an entire absence of voids. This resulted in some distortion of the forms, and considerable stiffening and bracing had to be added before the forms were able to resist the pressure of the concrete. The steel forms had undoubtedly justified themselves and given a smooth finish to the work. The 70-ton derrick had a range of 40 feet and was able to set two boxes in line ahead and one box at the side. The average rate of setting was four boxes in line per week, although this was exceeded towards the finish of the work. The setting was carried out under the personal supervision of the agent; it proceeded without any difficulty and fully justified the methods chosen. The longitudinal connection of the reinforced-concrete boxes, apart from the concrete core carried up from the cylinders to the top of the box, was strengthened by the upper deck, which was cast in situ, forming a continuous bond along the entire length of the wharf. The reinforced-concrete piles were assembled and cast upon an adjacent plot of land, being lifted as described in the Paper and transferred to site by rail. No difficulty was encountered in driving, which was all done by suspending the hammer from a crane without a pile-frame. Permission was sought and obtained to transfer a few of these piles to another work where difficulty was being experienced in driving reinforced-concrete piles through stiff boulder clay. The Trafford piles were driven through

Mr. Reinhold.

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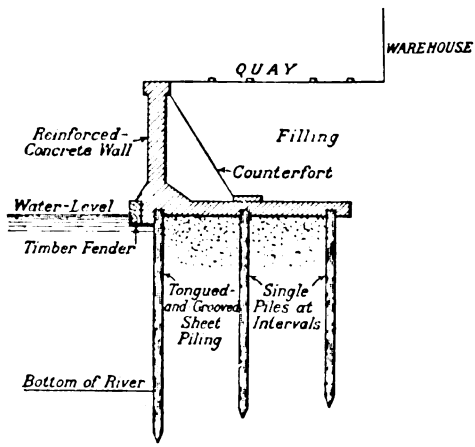
Mr. Reinhold. that ground without damage, although subjected to heavy punishment—a very severe test of their capabilities.

Mr. Saner. Mr. J. A. SANER congratulated the Author on the careful way in which the Paper had been prepared and on the interesting experiments and data which he had been able to describe. Mr. Saner's experience of the condition of old timber piles entirely agreed with what was found by the Author. On the River Weaver and elsewhere he had had considerable experience of such piling, and had always found that the portion under water remained good, whereas the life of the portion above water was seldom more than 25 to 30 years. There were many hundred yards of such work on the banks of the River Weaver, some of which had been in existence for probably 70 or 80 years, and the piles were still sound below water. So

Fig. 17.



confident had he been of the condition of some of these old piles that he had not hesitated to advise the replacement of the upper work with reinforced concrete, designed so as to rest upon and tie back the old work. That had been successfully done in many cases, and without a single failure, although some of the work had now been in existence for 15 to 20 years. In addition, an entirely new quay about 600 feet long had recently been built, under his advice, wherein the underwater work consisted of pitch-pine tongued-and-grooved piles with reinforced-concrete upper work, as shown in *Fig. 17*. He was not quite in agreement with the Author as to the cruciform piles mentioned on p. 70, as he thought they placed too great a stress on the sandstone rock into which they were driven. If the figures were correct, the 86 tons per pile meant approximately

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2·3 tons per square inch pressure on the sandstone, which was very high; but on the other hand a large part of the weight of the wharf and warehouse wall would be taken by the cylinders of concrete and spread over the adjoining soil, and to that extent the piles would be relieved. The piles, therefore, were more of the nature of stakes driven to prevent the slipping forward of the cylinders, and would so act until in the course of a very long time they rusted away. The Author had gone thoroughly into the question of driving, and Mr. Saner was unable to find similar diagrams in the "Minutes of Proceedings." This method of depicting the operations, so far as The Institution was concerned, might be considered novel and formed a valuable record and precedent. He was not very surprised, however, at the results shown in *Figs. 12, 13* and *14*, and he had always looked upon the pointed shoe as being more useful as a guide, and for pushing aside small stones, than as any appreciable aid to penetration. He agreed that 130 tons, as computed by the Wellington formula, was too much to allow on the piles; but, on the other hand, 60 tons appeared less than might safely have been put upon them, his experience being that Rankine's rule of 1,000 lbs. per square inch of the section of the pile—in this case about 80 tons—was a safe load when the pile was driven practically to rest and lateral movement was provided against. He thought it would be of practical interest to obtain data regarding (1) the relative frictional resistance to driving of two piles of the same sectional area and with similar shoes, one pile being made of reinforced concrete, and the other of timber, and (2) similar information for piles driven with the aid of a water-jet. There were in the "Proceedings" several references to driving piles through sand with the assistance of a jet of water. He well remembered when the embankment was made in front of Ellesmere Port on the Manchester Ship-Canal[1]. Unfortunately, no diagrams similar to the Author's were shown, but the results of applying a jet of water carried by means of a pipe alongside the piles was remarkable, and it certainly would appear worth while, when driving a number of piles, to make provision for a water-jet, especially as with reinforced concrete the pipe conveying the water could be cast in the pile and carried down to any desired depth, so reducing the risk of damage.

Mr. Saner.

Mr. E. G. WALKER considered that the design of this wharf was certainly unusual, and, so far as the immediate water-front was concerned, gave a very complete utilization of limited space. A remarkable feature was the extent to which the face of the quay

Mr. Walker.

[1] Minutes of Proceedings Inst. C.E., vol. cxxxi (1897), p. 39.

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Mr. Walker. was cantilevered beyond the front line of cylinders. The overhang appeared to be about 8 or 9 feet beyond the centre-line of the front row of cylinders. The only apparent reason for this arrangement was that there might have been a certain added convenience in the driving of the front row of piles from the old timber quay. It would appear, however, that it would have been better to set the piles farther forward; for in that event the cutting and patching of the lower ends of some of the cylinders would have been avoided. In addition, the improved width of the base of the structure might have been expected to impart additional stiffness, although, in view of the facts that the wharf and the foundations of the sheds formed a practically continuous unit and that all piling was carried down into the rock, this consideration was possibly only a secondary one. With regard to the use of mass-concrete foundations for the columns of the sheds, it appeared that those buildings were complete monolithic reinforced-concrete structures; and, as they were carried on reinforced-concrete piles, there would be every reason for considering the desirability of using reinforced-concrete bases as caps on the pile-clusters and so forming a continuous steel bond between the foundations and the superstructure. Prima facie, it would look as though the continuous reinforced-concrete design would be better and more economical, although he had come across other similar instances where the method used on the Trafford wharf had been adopted. It would be of considerable interest, therefore, if the reason for this procedure could be explained. Possibly the method adopted might work out more economically in regard to cost, although there seemed little reason to assume that such would be the case. The concrete seemed in general to be rather rich in cement, the ratios of cement to mixed aggregate being: for piles and reinforced-concrete cylinders, 1 : 4·70; for reinforced-concrete boxes, decking, floor-beams, etc., 1 : 5·23; for mass concrete, 1 : 6·25. In the absence of information as to the qualities of the materials and the magnitude of the unit working-stresses, it was difficult to criticize these figures.

The Author. The AUTHOR, in reply to the Correspondence, observed that in carrying out the work at Trafford wharf he had found that an opportunity presented itself to gather together information in regard to pile-driving which might prove of interest to the profession, and experiments had been made, with the assistance of the resident engineers and the contractors, with a view to extend the usefulness of the work. It was therefore gratifying to him that his efforts had evoked much constructive criticism.

The question of the loads which piles might be expected to sustain

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safely under various conditions was becoming one of increasing importance to engineers, especially since reinforced-concrete piling had come so much into use; and the suggestion by Mr. Latham that the subject should be considered by the British Engineering Standards Association, with a view to give the profession some pronouncement as to the correct formula to be used, appeared to be an excellent one, especially when it was remembered that so many formulas were now in being. As Mr. Ackermann had pointed out, Mr. Henry Adams examined the results of the application of thirty-nine formulas to a particular case, and found them all different. The Author had not used so many, but he found that nine formulas gave very various results. If the Engineering Standards Association decided to investigate the subject, it would no doubt be necessary to collect a very considerable volume of information respecting current pile-driving, and it would be of great assistance to engineers if a statement were issued giving the nature of the information desired by the Association. The Author.

Mr. Ackermann stated that, with the possible exception of Mr. Hiley's, there appeared to be no reliable formula in connection with any form of percussive pile-driving; but, as the Author had already endeavoured to show (p. 105), there was no doubt that even that formula must be used with very great caution coupled with practical experience. It should not be forgotten that the principle of introducing the modulus of elasticity into the formula, as done by Mr. Hiley, was adopted by Rankine as early as 1862.

He agreed that the dolly or packing on the top of the pile should be as small as possible. In recent work carried out by him the contractors—using a 3½-ton drop hammer—had reduced the packing to 3 inches in thickness, the pile-head being surrounded by a steel gland strap, 2 feet deep, in two halves bolted together.

He had for some time adopted closely-spaced helical hooping for a length of 2 feet 6 inches at the head of the pile to give additional strength, as indicated by Mr. Mitchell and used by Mr. Considère as early as 1910.

Mr. Halcrow apparently was not in favour of group driving. At Trafford wharf it had been carried out only experimentally and had not been used throughout the work. Unless it was specified, a contractor might be reluctant to use group driving, owing to the extra expense, especially if a pile-frame was in use with a drop hammer; and although at Trafford wharf the method had been found to give slightly better sets, it was doubtful whether the result would justify the extra expense incurred in moving the pile-frame to and fro.

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The Author. The cylinders had been originally designed to be in rings as mentioned by Mr. Latham. The contractors elected, however, to mould them in one piece, as being cheaper; it was certainly better in every way, and would be adopted in future work of a similar character.

Considerable difficulty had been experienced in carrying out reinforced-concrete work satisfactorily when it was only slightly raised above the water-level in the canal, owing to continual flushes from passing traffic and river-floods, which somewhat affected the water-level. It was essential that the subway for grain-conveying machinery should be perfectly watertight, although the floor was below flood-level, and this had largely influenced the design.

Although some boulders had been found in the clay in other places at the Manchester Docks, they were not in sufficient quantities to justify the description "boulder clay." There was no indication of boulders in the pile-driving records, and the clay would be fairly described as a stiff brown plastic clay containing very little sand—such a clay as would be used for brickmaking.

Mr. Latham's suggestion, that the weight of the pile would probably be eliminated as a factor from a future standard formula after investigation, was also the Author's view. The Author, however, could not agree that the weight of the pile could be altogether eliminated, and he agreed with Mr. Mitchell that there should be some specific relation between the weight of the hammer and the weight of the pile. The Wellington formula apparently presupposed a 1-to-1 basis; the Author's view was that it was desirable that the weight of the hammer should not be less than the weight of the pile to be driven, and it would be advisable to specify this to avoid misunderstanding with the contractor, who might wish to use available but unsuitable plant.

The front cylinders had been placed, as nearly as possible, centrally with the centre of gravity of the superimposed load. It would not have been wise to place them farther forward, with the resulting eccentric loading, having regard to the fact that, although the roof of the subway was constructed in situ and bonded the two lines of concrete boxes together, the floor was composed of pre-cast beams which were not bonded to the boxes.

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15 December, 1925.

SIR WILLIAM HENRY ELLIS, G.B.E., D.Eng., President, in the Chair.

The PRESIDENT announced that the Council had given considerable attention, in Committee and also in full Council, to the fact that the Abstracts of papers and articles in foreign periodicals dealing with technical matters were not receiving from the members of The Institution the appreciation which it was thought they deserved. The preparation of those abstracts was a costly matter to The Institution, and the Council were anxious that Institution money expended should benefit the members as a whole. The matter was one which affected the prestige of The Institution. The Council felt that work of a high order was being done, which brought credit to The Institution, and they were anxious that the members should more whole-heartedly appreciate the value of that work. The matter seemed to him to be of sufficient importance for him not merely to rely on the few words he had just said, and therefore he would read a short statement which the Council intended to bring before every Member and Student, in an endeavour to secure their earnest attention to the matter. During the Session he would have opportunities of alluding to the subject, also at the gatherings of the Local Associations, all of which he hoped to attend.

[The President then read a statement which was subsequently issued to all Members and Students of The Institution, in the "Sessional Notices" (Session 1925–26, Part 3, p. 47).]

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

(*Paper No. 4540.*)

"Pressure-Boosting Station for the Waterworks of the City of Monte Video."

By ARTHUR HONEYSETT, Assoc. M. Inst. C.E.

THE city of Monte Video has had for many years past to meet a constantly increasing demand for water due to the growth of its population, and although this increase was for a time fully provided for by enlargements of the existing works, such as the

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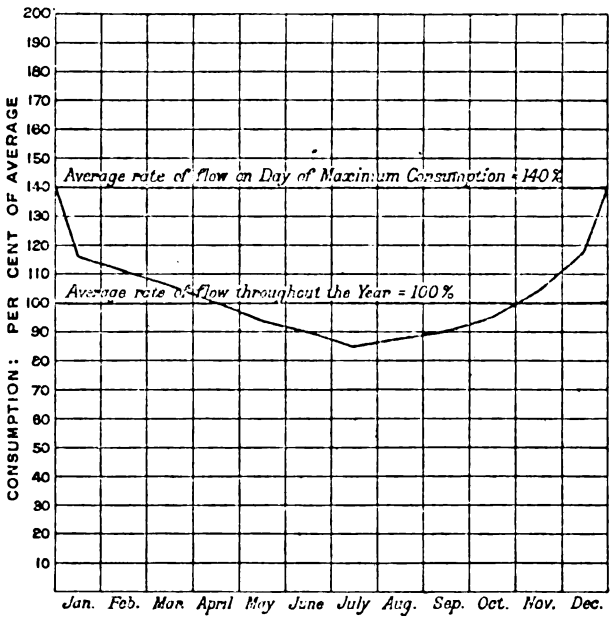
addition of new pumping-engines, reservoirs, and main pipes, yet a point was at length reached when these methods could no longer be satisfactorily pursued, and something of a more radical nature became unavoidable. The object of this Paper is to describe the new devices which have been introduced.

Water-Supply.—The water is drawn from a river at a point situated about 35 miles from the city, whence it is pumped, after purification, to reservoirs constructed upon high ground 12 miles from the centre of the city. These reservoirs were constructed with a capacity sufficient not only to equalize the flow of water during

Fig. 1.



the 24 hours, so that pumping and filtration could proceed at a uniform rate, but also to provide for complete cessation of pumping for several days in emergencies. By increasing the capacity of these reservoirs, by the installation of new engines, and by increasing the number of pumping-mains and of pipes connecting the reservoirs with the city, it was possible for many years to keep pace with the demand, which for a long period has shown an average increase of 500,000 gallons per day each year.

Between the river and the reservoirs this increase has been, and continues to be, met by simple additions to the pumps and pumping-

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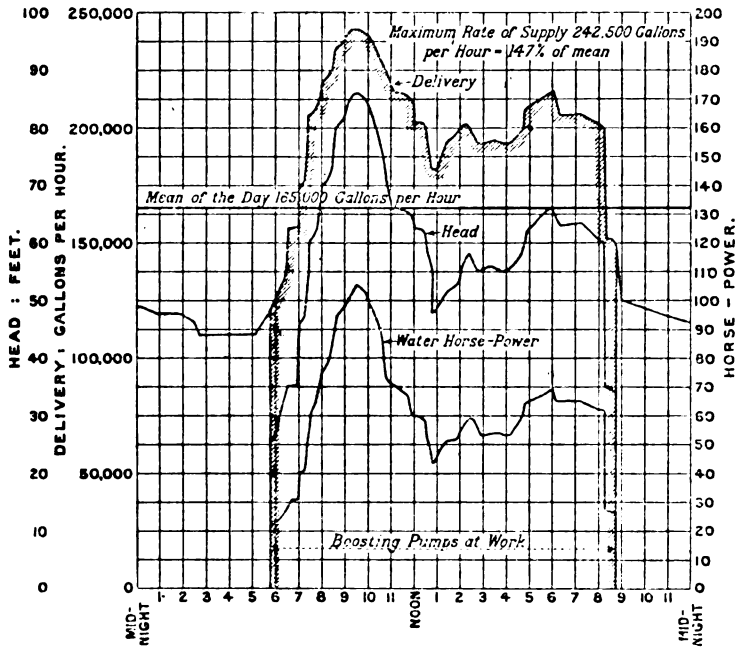
mains, but from the reservoirs onwards the problem is much more difficult, for the following reasons. The climate of Monte Video consists of a cool winter followed by a hot dry summer, resulting in a marked difference of consumption between the mean summer and winter supply, as is shown by *Fig. 1.* This diagram, which is based upon the average of 12 years' observations, shows the variation of the mean demand for water throughout the 12 months of the year. While the rate of consumption in December exceeds the mean by 18 per cent., that of July falls short of the mean by 15 per cent., so that

Fig. 2.



the December consumption exceeds the July by 39 per cent.; and further, on a particular day near the end of the year, the demand shows a further increase of 20 per cent. above the December average, the result being that on one day in the year 40 per cent. more water may be required than the mean and 65 per cent. more than the minimum. In addition to this, provision has to be made for the variation which occurs during the 24 hours of the day itself. *Fig. 2* shows the variation as actually recorded during one day of maximum consumption in the early part of 1925, indicating that for about 1 hour the demand stood at 47 per cent. above the mean for the day.

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It is evident, therefore, that, since no storage existed in the town itself, it was necessary for the pipes leading from the reservoirs (hereinafter called the distributing-pipes) to provide for a flow 2·05 times the yearly average, and it is further known that this latter amount is itself increasing at the rate of approximately 15 per cent. per annum. The increased duty thus thrown upon the distributing-pipes was for some years met by additions to their number, until at last four pipes had been installed, three of 18 inches diameter and one of 24 inches; and the demand was thus met until 1914, when lack of pressure in the higher parts of the town commenced to become imminent.

A temporary improvement was effected by constructing further reservoirs at Las Piedras, having their water-level 30 feet above the ground, and using these for the supply of the higher parts of the town. Although this course gave temporary relief, it could not be regarded as very satisfactory, for the following reasons:—

1. It was expensive and the construction of such reservoirs could not be continued indefinitely.

2. The necessity of reserving each pipe for the exclusive use of a particular zone of the city prevented one pipe from supplementing another, so that full advantage could not be taken of their capacity as a whole.

3. The extra pressure could be obtained only by keeping the reservoirs full; they therefore lost their utility as regulators of the pumping, as, while kept full, they could no longer take care of the peak load, the result being that, when most needed, an expensive reservoir was only performing the duty of a stand-pipe.

The peak load was reduced to some extent by the construction of another reservoir on high ground (but less elevated than the main reservoirs) close to the city. By filling this reservoir at night and using it to take the day load of a low-lying portion of the city the maximum flow in the distributing-mains was reduced and the conditions were slightly improved; but, as this reservoir is small and its extension difficult, the improvement was not great, and was soon neutralized by the continued growth of the city. In these circumstances the laying of a fifth distributing-main appeared inevitable.

For obvious reasons nothing could be done during the war, and on the re-establishment of peace the cost of material had become so excessive that the work had again to be postponed.

In the meantime a new factor arose which materially increased the difficulties, namely, that with the growth of population a spreading of the inhabited area on to higher ground had resulted,

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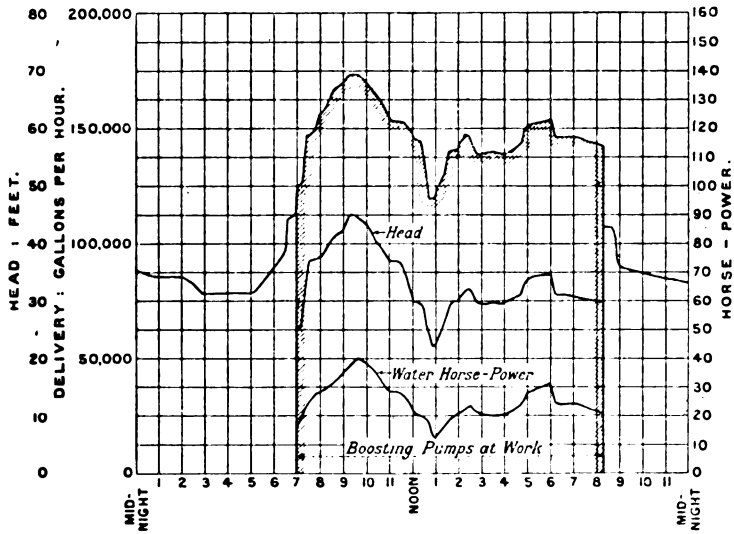
and along with this there was a considerable increase in the height of new buildings. Thus not only had the pipes to carry more water, but the head available was continually diminishing; and, irrespective of the high cost of a new pipe, the conveyance through it of a large quantity of water with a vanishing head was in all ways uneconomical. Such a pipe would at that time have cost at least £250,000, and its utility would have been only temporary.

At length, in 1920, the matter was placed in the hands of the late Hon. R. C. Parsons, M. Inst. C.E., the consulting engineer to the Waterworks company (whose assistant the Author then was); and after much consideration and searching for sites for possible

Fig. 3.



service-reservoirs, it occurred to the latter that the most economical method would be to abandon for the time being all idea of increasing either the number of pipes or the capacity of the reservoirs, and to construct near the latter a new pumping-station by means of which additional pressure could be placed upon the distributing-mains.

Character of the Boosting Load.—The lay-out of the existing piping indicated as most convenient a site for the pumping-station near the reservoirs, but about 30 feet lower. This was suitable for the installation of centrifugal pumps, as they would be naturally primed. At first an attempt was made to design a station having steam, gas. or oil, as its motive power; but the result was by no means

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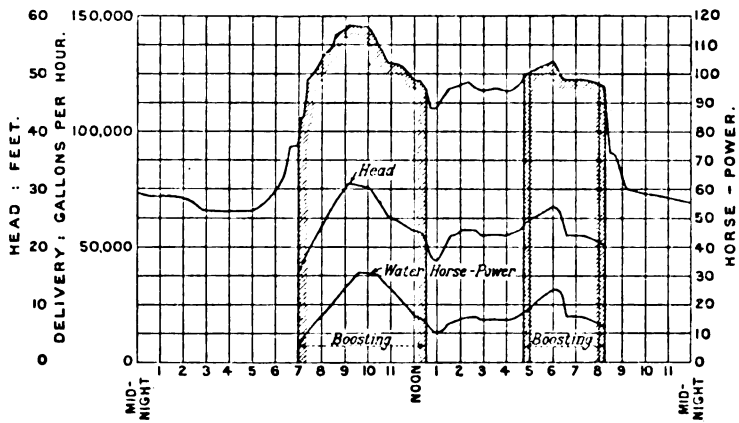
satisfactory owing to the intermittent and variable character of the load. A study of the characteristics of this load is necessary for an understanding of the nature of the problem. At the time mentioned the capacity of the distributing-mains under their gravitational head was approximately equal to the mean rate of flow to be anticipated during the next few years, so that only the peak load had to be taken by the pumps. *Fig. 1* shows that for seven months of the year no boosting would be required under normal conditions, although at any time there might be exceptional hours when the gravitational head would be insufficient. Conversely, during the remaining five months there might be days such as those of abnormally low temperature or heavy rain when the demand for water would be so restricted that boosting might

Fig. 4.



be dispensed with. It was further anticipated that, even upon a day of maximum demand, there would be several hours of the night when no boosting would be necessary, and that even when it was taking place the additional head required by the pipes was likely to be highly variable.

These anticipations have in the main been fulfilled, but as the scheme has proved entirely successful and has induced, through that success, a considerable increase in the quantity of water consumed, the idle periods have proved to be much shorter and less frequent than was originally anticipated. *Fig. 2* shows the variations in discharge from the reservoirs through the 24-inch main during the day of maximum consumption in the summer of 1924–5, plotted from the record of the self-registering meters

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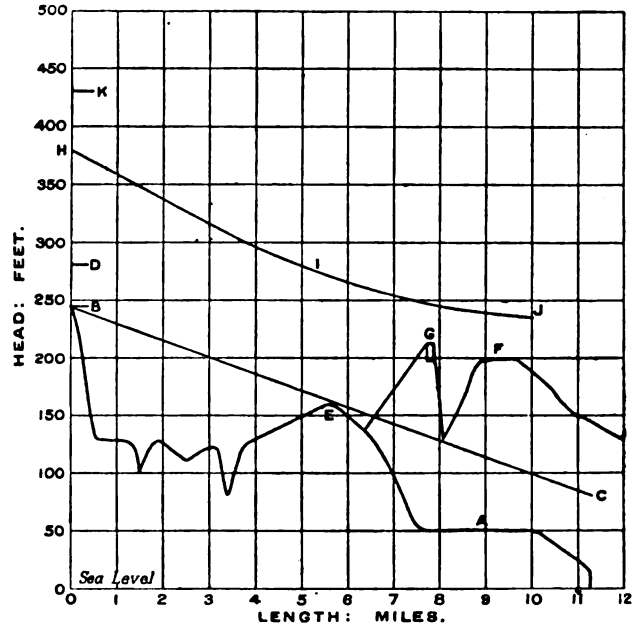
for that main. *Figs. 3* and *4* show the variations in discharge of this same pipe reduced in scale so as to represent respectively an average day and a day of minimum consumption during winter. As this pipe discharges about one-half of the total quantity of water, it may be taken as fairly representing the consumption of the city as a whole. Thus there is even now a considerable interval during which no boosting is required, although this is much shorter than it was 5 years ago. It is estimated that at the present rate of increase the boosting-plant will have to be con-

Fig. 5.



tinuously in action in about 20 years; but long before that time boosting will have become unremunerative, and the capital expenditure on a new pipe-line will have become justified. In the meantime the boosting-station will have saved many years' interest upon the cost of the main, and will permit the main, when laid, to be of smaller diameter than would otherwise be the case.

The problem is perhaps more clearly shown in *Fig. 5*, which represents the lengths and levels of one of the distributing-mains. Line A shows the level of the part of the town for which the works were originally designed 50 years ago. Point B

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is the level of the bottom of the reservoirs, and was sufficiently elevated to supply the area at level A through the 18-inch pipe originally laid, the hydraulic inclination being that given by the line BC. When this inclination was found insufficient to convey the increasing quantity of water, other pipes were laid to supplement the original one until, the last of these proving insufficient, the reservoirs were raised to the level marked D, so as to get sufficient pressure for the proper supply of the city, the populated part of which had by that time begun to extend towards the higher levels E and F. This raising of the water-level at D was supplemented by the erection of a small service-reservoir, constructed within the limits of the city, as shown at the point G. This supplies the old low-lying level A, and, being filled at night, reduces the quantity which the mains have to deliver during the day, thus making the head upon them more effective.

The foregoing represents the position when the works now being described became necessary, and it will be evident that reservoir G could not supply the houses which were beginning to be built at F, and that the older reservoirs at D, situated 8 miles away, were very little better. At the present time it is found that the hydraulic gradient H I J is necessary to maintain a supply throughout the city at a reasonable pressure, the height from D to H, 100 feet, being the amount which is now added by the boosting-pumps. This amount will increase year by year until the level K is attained, which represents the limit of the pressure for which the boosting-station is designed.

Figs. 2, 3, and *4* show, in addition to the quantity which has to be pumped, the head, above that available by gravitation, required to pass the water through the pipe, and also the resulting water horse-power. The latter is highly variable. For 9 hours during the night no pumping is necessary, and when the pumps start the power required is only 20 HP. It increases, however, rather rapidly during 4 hours, to 100 HP. It then falls for the next 3 hours, to 45 HP. Then follows 7 hours at an average of 60 HP., finishing with a sudden fall to 25 HP. This represents the condition which prevailed upon the day of maximum consumption; but *Fig. 3* shows that the power required is, as a rule, only 40 per cent. of the above figures, while *Fig. 4* indicates that upon some one day of the year it may be expected to fall still lower, the power being at one time only 7 HP.

These diagrams represent about one-half of the total work to be done by the boosting-station; and allowing for this and for the increase expected in the next few years, it was found necessary to

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design a pumping-plant which would deliver as and when required 200,000 to 500,000 gallons per hour against a head ranging from zero to 150 feet and absorbing a net power ranging from 15 to 375 W.HP.

Choice of Pumping-Installation.—To ensure reasonable economy in such circumstances it was necessary to exercise so effective a control that the output of the machinery should follow closely the demand as it changed from moment to moment; and further, that the machinery should be at work only at times when the extra pressure was required. The times of stopping and starting were quite uncertain, depending very largely upon the weather. These considerations indicated the advisability of some form of automatic control, as did also another reason independent of economy, for it was evident that it would have been unsafe to add so largely to the pressure upon the old pipes and fittings in the lower part of the town. The maximum pressure could be placed with safety upon the pipes only when the flow of water in them was sufficient to absorb it in friction. Every diminution in flow must therefore be accompanied by a corresponding diminution in pressure.

These conditions pointed to the desirability of electricity, drawn from some large public source, as the motive power to be employed. This would permit of automatic starting and stopping of the pumps and regulation of their speed, so that the pressure produced and the energy consumed at each moment would not exceed that required by the work to be done, while at the same time stand-by losses would be reduced to a minimum. Some hesitation was naturally felt in adopting, for so important a public service as water-supply, a source of power not directly under the control of the persons responsible for maintaining the service; but this disappeared in view of the fact that the electric supply was under the control of a government department. Thus the consumption of water and the supply of the power necessary for securing that consumption were both ultimately in the same hands.

The Author circulated a general specification of the conditions to be fulfilled among the chief manufacturers of electrical machinery in Great Britain, but the result was highly disappointing. A large majority of the replies consisted of recommendation of some standard piece of apparatus, with complete disregard of the special conditions specified; and only two of the replies showed any signs that the matter had been properly studied. Of these two the more complete, and by far the cheaper, was that presented by the Brush Electrical Engineering Company, Ltd., of Loughborough, who ultimately obtained the contract; and it gives the Author much

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pleasure to testify to the great care and skill and the continual interest manifested by the technical staff of that company; and with them he is pleased to associate the name of Mr. E. N. Bray, who was sub-contractor for the special switchgear.

The economical nature of the plant supplied was due in no way to any inferiority in quality or capacity, but solely to the fact that the designers had taken the trouble to work out the best simple method of securing the desired end. It was, in fact, a triumph of engineering over shop-keeping.

Design of Pumping-Plant.—With the help of the electrical methods thus suggested, the Author was able to work out a complete scheme, as shown diagrammatically in Fig. 6, Plate 5, and in general plan in Fig. 7, Plate 5. Fig. 6 shows that the installation may be divided into two separate portions, the unit marked A being distinctly different from units B, C, and D. This is due to the fact that the latter units were the first to be installed and are those to which the above remarks more particularly refer.

As was perhaps natural, there was in the early stages of the design a tendency to rely as little as possible on electricity, and, whenever a hydraulic or mechanical device could be employed for any desired end, to give it the preference; it was found, however, that in the delicate controlling operations which had to be effected such generally neglected factors as the viscosity and surface tension of water, or the backlash of gearing, were sufficient to prevent a really satisfactory result. The consequence was that these hydraulic and mechanical devices were one by one replaced by purely electrical appliances, and the diagram represents the final result, showing the installation as now at work.

It was due to the desire to proceed with caution that the size of the first three units was limited to 100 HP. and the additional head restricted to 100 feet. The unit subsequently added is considerably larger. The form and dimensions of the boosting-station are shown in Fig. 7, but its action will best be followed from Fig. 6. All the water flowing to the town is made to pass through the boosting-station, being brought into it by the pipe F. This is 33 inches in diameter at the right-hand end, and diminishes to 18 inches at the left-hand extremity, where it terminates in a dead end. Parallel to this is laid another pipe H, partly 18 inches and partly 24 inches in diameter, placed in communication with pipe F by means of five cross pipes, G, each 18 inches in diameter (only four of these are shown in Fig. 6). When the gravitational head is sufficient to produce the required discharge, the water flows through these cross pipes into H, and thence into each of the distributing-mains I, J, K, and L, each main taking as much as its district

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demands. When the demand becomes too great to be supplied by the gravitational head, the pumps are brought into use automatically by the following means. On each of the distributing-mains is placed a venturi tube (58 and 59),[1] the differential pressure in which is not only used for recording the flow in the usual way, but is also carried to a specially-designed mercury switch (57), consisting of a pair of iron cylinders mounted on a rocking lever and connected by a small pipe. The cylinders are partly filled with mercury and are connected one with the throat and the other with the upstream end of the venturi tube. As the differential pressure increases it forces some of the mercury from one cylinder to the other, thus upsetting the balance of weight and causing the lever to tilt, closing a mercury contact upon the electric circuit and thus starting the pumping-machinery. The amount of venturi head required for this purpose can be adjusted by means of a sliding balance-weight, so that the start can be arranged to take place at any desired flow through the distributing-main; and a sufficiently quick make and break is secured by adjusting the quantity of mercury, and hence altering the level of the centre of gravity of the moving parts relative to the centre of movement. A potential of 440 volts and a current up to 6 amperes are successfully dealt with by this apparatus. Immediately this tilting occurs, the pump starts, and, running quickly up to its minimum working-speed, puts an additional head upon the main. For practical reasons connected with the design of the motor this additional head could not be made less than 30 feet; this amount is therefore added in one step to the head upon the main, which is thus given the possibility of discharging an increased quantity of water corresponding to this addition to the head.

No further change takes place in the speed of the pump until this head is exhausted, when another piece of apparatus is brought into play to produce the necessary further acceleration. This consists of a mercury U-tube (64) built up of a number of thin perforated steel disks separated by insulating washers and connected to the venturi tube in the manner already described. Each disk is electrically connected with a resistance unit so arranged that, as the mercury is driven down in the tube by the increase of the venturi head brought about by any increased flow in the mains, resistances are successively cut out and the speed of the motor is increased. By properly proportioning these successive changes of resistance, the speed of the pump can be regulated so as to follow exactly the demand made upon it by the changes required both in delivery and head. The mercury in this U-tube is not acted upon directly by

[1] The figures in brackets refer to the numbered details of Fig. 6, Plate 5.

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the water, columns of oil, contained in the two small reservoirs (b), being interposed between the two, so that any electrolytic action is avoided.

When the demand for water begins to fall off, the reverse action occurs; the venturi pressure falls in (64), the resistances are re-introduced, the speed falls, and ultimately, when the pressure becomes insufficient to counterbalance the weight of (57), the mercury switch opens and the electric machinery comes to a standstill.

As the speed-regulating apparatus has no moving parts or inertia except that of the small quantity of mercury in the U-tube, whose movement is exceedingly slow, it may be regarded as being entirely without friction. The falling curve connecting pressure and speed coincides exactly with the rising curve. The only lag observable is that due to the friction of the bearing about which the lever of (57) turns; this causes the current to continue until the flow of water in the mains falls slightly below that which caused the machinery to start, the difference being about 2,000 gallons per hour. Experiment showed that so good a result could not be approached with any mechanical or hydraulic apparatus that could be devised.

The Pulsometer pumps are of the standard C type, having 12-inch suction and 10-inch delivery. On test they gave a maximum efficiency of 78 per cent. at 500 r.p.m. with a delivery in each one of 1,700 gallons per minute against a head of 62 feet. When absorbing the full amount of power they will discharge 2,200 gallons per minute with an efficiency of 75 per cent. against 100 feet of head, and require a speed of 650 r.p.m. Since the pumps generally work at about half load, the position of the point of maximum efficiency upon the pump characteristic curve may be considered very satisfactory. At the lowest speed at which they are intended to run, 385 r.p.m., their efficiency is 72 per cent. and their delivery 1,300 gallons per minute against a head of 40 feet.

Considerable experience having now been obtained concerning the actual friction in the distributing-main, which was not accurately known at the time the work was designed, it is intended to modify these pumps so that 100 HP. will be employed in delivering a somewhat smaller quantity of water to a head of 150 feet instead of 100 feet; by this means, although the efficiency of the pumps will be slightly reduced, the overall efficiency of the station will be increased, since it will more nearly conform with the actual conditions revealed by experience.

The establishment of the boosting-station has enabled the friction in the four distributing-mains to be measured with considerable accuracy. To do this the quantity of water passed by each of the

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venturi meters was noted at ½-hour intervals throughout a day. The pressure produced by the pumps was also observed, and similar pressure-readings were also taken at a point on each main as far removed from the boosting-station as was possible without the results being vitiated by changes of flow. From these observations a formula for the discharge was calculated which was found to assume the form of $v = c\, r^{\frac{1}{2}}\, i^{\frac{1}{2}}$, the coefficient c having the following values :—

Main No. 1	50 years old,	c =	71
„ „ 2	40 „ „	c =	72
„ „ 3	30 „ „	c =	86
„ „ 4	20 „ „	c =	79

These figures are considerably lower than is generally assumed to be correct, and the head required is higher than was originally anticipated. This is not due to any deterioration of the pipes with age, as when opened out they were found to be quite clean and free from incrustation.

Detailed Description of the Electrical Equipment.—The main conductor (Fig. 6, Plate 5) enters the building through a chamber (121), in which are placed various safety devices such as lightning-arresters, overload cutouts, and control-meters, installed by the Electric Supply Administration for its own satisfaction and supplementary to similar appliances installed in the main building by the waterworks company. From here the 6,000-volt three-phase supply passes into the boosting-station building to a main switch (66). This is a standard G.E. 600-kilowatt high-tension cubicle containing an oil switch, overload trip-gear, and volt-, ampere-, and wattmeters, the last being provided with a self-recording chart. From this main cubicle the current is distributed to three similar cubicles of smaller capacity, each of which controls one of the sets of pumping-machinery.

The voltage is stepped down by static transformers (68) to a working pressure of 440 volts, and current at this potential is carried to three-phase induction motors (70) of the slip-ring type, having a standard Allen West starting-panel (69).

This panel contains four contactors ; one closes the stator circuit, and three cut out successive steps of rotor resistance. All are worked by electro-magnets, the first of which is actuated by the current passing through the mercury switch (57) already described, the others being brought into use successively after predetermined time-intervals by the action of their predecessors.

On to the shaft of the rotor (c) of the motor (70) is keyed the armature of a direct-current generator (d), which under the

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conditions prevailing when the machinery starts produces a current sufficient to drive the pump motor (72) at its minimum speed. The field necessary for this current, as well as that of the pump-motors, is produced by a separate exciter (e) driven by the same shaft, and its amount is regulated by a hand-adjustable resistance-board (71). The pump (75) can therefore be set to run at such initial speed as may be desired, within the capability of the motor.

It is evident that this speed should be as low as possible, for at the moment of starting only a very small additional head is required in the mains; but as, in view of subsequent operations, a very low speed would require a pump-motor of excessive size, a compromise has to be effected, and, as has been stated, the first step of the speed is such as to produce an additional head of 30 feet, this being the condition which gives a motor of practical size with only negligible loss of efficiency.

It will be observed that the pipe H is provided with a number of valves, there being one on each side of each pump; these are normally kept closed, so that at the moment of starting the pump works against a dead end and no water is delivered. As soon, however, as any current is produced by the exciter (e), this current passes through a solenoid (56), and commences to open valve (54) by means of a hydraulic relay (262). The acceleration of the motor (72) and the opening of the valve (54) continue therefore simultaneously, so that by the time the motor has attained its minimum working-speed, the valves (54) are fully opened and the water delivered by the pump is free to pass into the distributing-mains. The extra pressure now existing in pipe H causes the closing of the reflux valves (37) and (53), and prevents the water from flowing backwards through the by-pass pipe G, or through any pump which may not yet have started revolving. The rate of opening of valves (54) and the rate of speeding up of the pump can be regulated so that sufficient time elapses to prevent these reflux valves from closing with too great a shock.

When the pressure required in the mains exceeds 30 feet, a further increase in the speed of the motor (72) is produced by increasing the voltage generated in the generator (70d); this is done by cutting resistances out of its field. The resistances are mounted on a panel (73), and are cut out by the movement of the mercury in tube 64 consequent upon the gradual increase of the venturi head.

This panel contains two sets of terminals, the space between which forms a break in the continuity of the conductors connecting the resistances with the mercury tube. By suitably bridging across these two rows of terminals, the pressure-quantity curve given by

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the upper row can be made to coincide with the pressure-speed curve of the lower one, and thus the pump can be made to accelerate and decelerate with any desired degree of accuracy; further, the solenoids (56) can be brought into action at any moment, and, if required, a new pump can be started by the action of a venturi tube remote from it. Thus a great variety of conditions can be met, and, when adjustments have once been made to suit the conditions found to prevail, the automatic action ensures that they will be properly met every time they recur.

For purpose of experiment and investigation hand starting-switches are provided on panel 69, and a revolving arm (73b) enables the movement of the mercury in the U-tube to be imitated by hand at any time. These appliances have been found of great service in making the various adjustments. Excessive currents in either the armature or the field of the pump-motor are prevented by a resistance and fuses (70f) and (70g).

A valuable feature of this method of control is that the appropriate magnetic field is always in existence in the pump-motor, so that sparking is avoided; and as the generator only builds up its voltage as required, it is possible to connect the generator to the motor without any starting-resistance.

The work described was completed and brought into service towards the end of 1922, and its advantages, as regards the improvement in the water-supply, were immediately apparent. It had been expected that the demand upon the boosting-plant would be small at first and that a few years would elapse before its powers would be fully called upon; but so much advantage was derived from its use that when, early in 1923, the control of the work passed into the hands of the Author through the sudden death of Mr. Parsons, the first problem he encountered was to provide an immediate increase of power.

At that time the output of the boosting-station was determined by the capacity of the electric mains, which, where they formed part of the general conductor system of the town, had a capacity which only permitted the boosting machinery to work at two-thirds of its full capacity. As the increase of this section of conductor was likely to be a matter of considerable expense and delay, the Author resolved to obtain an immediate increase of power by improving the power-factor of the machinery. He recommended, therefore, the addition of condensers (250) to the a.-c. motors. The effect of their installation was a rise of the power-factor from 0·85 to 0·97, producing an addition of 30 HP. to the power of the station. The use of condensers for this purpose, in preference to other

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methods which were investigated, was decided on because they were found to give an equally good result at a much lower cost in capital expenditure, coupled with an entire absence of working-expenses.

This, however, was only a temporary measure, and the reinforcement of the conductors was proceeded with as quickly as possible, with the result that, by the end of 1923, the full power of the station, namely 300 HP., was available. The effect of this was that the discharging-capacity of the four distributing-mains was increased by 2,000 gallons per minute—equivalent to the laying of a new 24-inch pipe—the additional cost having been only a small fraction of that of such a pipe.

Mercury-Rectifier Type Unit.—It had been the original intention to add further units of 100 HP. each, as they might be required, but the experience so far gained showed that with the addition of one such unit the power would soon become inadequate. The Author, therefore, decided that the next unit should be of 300 HP., so as to double the capacity of the station at one step, which would enable an addition of 1,750 gallons per minute to be made to the discharge, and would necessitate raising the boosted head to 150 feet.

By this time certain extensions and changes had been made to the town piping, with the result that it was most effective to concentrate the power of the new unit entirely upon the 24-inch main (I, Fig. 6, Plate 5), leaving the three 18-inch mains (J, K, and L) to be worked under the original pressure by the 100-HP. units, the full interchange-ability of the pumping units and the mains being, however, preserved as before.

The only point upon which the first three units might be criticized was their efficiency, which, as is the case with all motor generators, was not very high. The overall efficiency of the machinery at full load was 71 per cent., and at half load, which represents more nearly the average working condition, it was only 63 per cent. The power so lost represents, when capitalized, about £10,000, and it appeared worth while to seek a method by which some portion of this could be saved without in any way sacrificing the certainty and accuracy of the automatic control. This latter consideration confined the choice, after adequate inquiry, to some form of conversion of alternating to direct current. Of methods of doing this, other than by motor generators, only one could be found which would fulfil the conditions, namely, the use of mercury rectifiers. A contract was therefore placed with the Hewittic Electric Company, Ltd., for a set of four 150-ampere rectifying bulbs which, when arranged to give a potential on the direct-current side of 500 volts, would produce the required power.

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The selection has been fully justified, since at a cost no greater than that of motor-generators the efficiency of conversion at half load is 89, instead of 71 per cent.; the overall efficiency under average working conditions has been thus raised to 78 per cent., and a saving of more than £5,000 has been effected in the capitalized value of the lost energy.

The effect of the changes introduced into the original plan is that the main conductors (an expensive item, as they are 12 miles long), which were originally considered sufficient for only 500 HP., are now capable of supplying enough current for 740 HP., while the building, another expensive item, will accommodate this latter power and a further 300 HP. as well. The economy corresponding to these changes may be placed to the credit of the condensers and the mercury rectifiers, and covers their total cost several times over.

As the power required is such that the mercury rectifiers consisted of necessity of several rectifying units, and as the largest unit of a suitable kind available at that time was of 75 HP., the division of the rectifier into four parts was at once suggested. It was further found to be of practical convenience to group these four parts into two groups of two each so as to form two independent rectifiers of 150 HP. each.

Since for some time there would be no spare plant corresponding to this half of the installation, this division of the rectifier was maintained throughout the whole of the appliances connected with it, so that pumping unit A is really served by two entirely separate rectifying units; thus in case of repair being required by any part of the rectifiers, only one-quarter instead of one-half of the whole power of the station is thrown out of service, while the failure of a rectifying bulb reduces the power of the station by only one-eighth.

The incoming panel (66) is connected with two distributing-panels (244)—corresponding to the panels (67) for the other pumping-sets—each of which controls a transformer (245). These transformers, by the Hackbridge Electric Construction Company, are of special type adapted to the requirements of the rectifiers, and each has two complete circuits upon the secondary side. Thus, when the two rectifiers are working together, the alternating current is supplied through twelve separate conductors. The transformers reduce the voltage from 6,000 to 769 volts, the latter figure being that required to give by conversion 500 volts direct current.

The conductors are carried to a twelve-pole switchboard (246), the twelve contactors on which are capable of being closed by the four electromagnets (c). The switchboard is provided with isolating-switches (255), so that the twelve contactors are separated into two groups, either or both of which can be used at one time, and with a

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push-button switch (251) for starting or stopping the rectifiers by hand when required. In practice, however, starting is effected automatically by closing the solenoid circuit by the action of one of the mercury switches (57) already described.

As the secondary voltage of the transformers would be inconveniently high for working the solenoids, a small transformer (249) is introduced, producing a further fall of potential to 220 volts and supplying current for working the switchboard. Only one of these small transformers is installed, but it can be placed in communication with either of the main transformers by means of a change-over switch (256).

The action of the rectifiers (247) is in itself very simple. They consist of an exhausted glass bulb having a small pool of mercury in the bottom. The alternating current enters through the anode terminal, forms an arc between it and the cathode terminal, and leaves the latter converted into direct current. Unfortunately, many subsidiary operations have to be performed, so that the rectifier as a whole becomes rather complicated, as is seen from Fig. 6. Each of the four bulbs is in a separate cubicle, those marked (1) and (2) forming a complete rectifier, while (3) and (4) form a separate and precisely similar one. The conductors from the main switch terminate in one of the cubicles of each rectifier in a six-pole switch (a); these switches are kept permanently closed, whether the rectifier is in operation or not, their only use being to facilitate the working of one of the rectifiers when the other is put out of service.

A potential having been thus established in the alternating side of the rectifier, a small subsidiary current passes through a transformer situated at (b), the low-tension current from which passes first to a fan (c) for cooling the rectifying bulb (i), and secondly to the starting-gear (d).

This gear consists essentially of a solenoid which, when actuated by the above-mentioned current, causes the bulb to tilt slightly, so that the pool of mercury flows to the left-hand side and establishes electric communication between the anode terminal (k) and a secondary cathode terminal (q), causing a momentary current to pass through q back into d, the effect of which is to neutralize the tilting current. The bulb then falls back, breaking the mercury contact with q, but retaining the current in q by means of an arc, which is thus struck between k and q.

The heat produced by this arc vaporizes the mercury, which, rising to the level of the terminals (u), causes further arcs to be established and actuates other solenoids in (d), the object of which is

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to undo the work done by the first solenoid and place the apparatus in such a condition that it can be stopped and started again automatically when required. This arrangement is somewhat complicated, but is highly ingenious and reflects great credit upon the designers of the rectifiers.

The continued vaporization of the mercury causes the main arcs to be established between the cathode terminals (j, j, j, and k), so that the main three-phase current entering through the former leaves through the latter in the form of a continuous current.

To ensure steady working, the three-phase current before entering the rectifier passes through a set of inductance coils (e and f), which, when brought into use by the switch g, also serve the purpose of balancing the load when one of the bulbs is out of service.

The action of the rectifiers is further steadied by choking-coils (o) on the d.-c. side. This side of the rectifier is also provided with ampere-, and voltmeters (l and m) and an overload cut-out (n).

From the d.-c. side of the rectifiers conductors are carried to a terminal board (258), from which the direct current is distributed to the pump-motor and to such other points as may be required. In order to extend the duplication of the plant as far as possible, each rectifier is furnished with a separate conductor, and a separate return is carried to the neutral point of each main transformer; thus the duplication is carried out right up to the terminals of the pump-motor.

The Brush pump-motor, which is of 300 HP., showed, when tested, a speed-variation from 800 to 290 r.p.m., a range considerably beyond what will ever be required in practice. The motor is started through a set of resistances which are cut out by the closing of successive contacts controlled by an automatic switch and time lag.

The automatic speed-control of this motor presented some problems which were different from those of the smaller sets. In the latter, as has been described, the change of speed was effected by variations in the voltage of the direct current supplied to the armature, and this had worked so well that the Author was anxious to employ the same method in the new unit. Its application was difficult, however, owing to the characteristic differences between a mercury rectifier and a motor generator. The only method of obtaining the result appeared to be to vary the secondary voltage of the transformers. In most rectifiers this is done by the provision of certain additional attachments, but it could not be so arranged in the present case. Since a range from 800 to 290 was far greater than anything attempted so far, the apparatus

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required would have been quite novel; and, although the end did not seem impossible of attainment, yet the delay likely to arise in designing and experimenting with a novel piece of machinery could hardly be afforded. The Author was therefore reluctantly obliged to fall back upon the more conventional method of controlling the speed by means of variation in the strength of the field. The arrangement was worked out in detail by the Brush Electrical Engineering Company and Mr. Bray.

The motor (240) is provided with two separate field coils; one of these (a) is in series with the armature, and the other (b), which is shunt wound, acts in opposition to the former; the effective field is therefore the difference between the two, the shunt field being considerably the larger. There are two distinct means of varying the field-strength, one by hand and the other automatically: although the latter is used in practice, the former method is of considerable importance, as it is relied upon to replace the automatic method should this be out of use for any reason, and also to actuate the machinery occasionally during any period of idleness, in order to keep it in proper working order.

The use of one or the other method is determined by the position of a link (252) upon the d. c. starting-panel; when this is joined up in the left-hand direction from its centre contact, as shown in full lines, it places the automatic regulator in action; when to the right, as shown in dotted lines, the hand regulator is at work.

Assuming it to be in the latter position, it will be seen that the field coil (b) is a simple shunt across the armature of the motor (240), with no external resistance; and at the same time, if the motor be regarded as having only just started and doing its minimum work, the armature current, and consequently the series field, is a minimum. The resultant difference between the two fields is a maximum and the speed of the motor therefore a minimum.

Part of the complete shunt field circuit consists of a series of resistances (243) which can be inserted by a radial arm passing over contacts in the usual manner. These are isolated when the machinery is stationary by a switch (269), which closes by a spring as soon as the machinery stops; and, as no appreciable resistance is offered so long as this switch is closed, the motor must always start with its full field and at its lowest speed, no matter what may be the position of the radial arm (243).

When it is desired to increase the speed of the motor it is first necessary to move the radial arm to its zero position; it then mechanically opens the switch (269), and the current, instead of passing through the switch, goes to the centre of the disk and back through the radial arm. No perceptible addition is made to the

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field resistance, as the radial arm is at this moment, of necessity, at zero. When the radial arm is moved back from zero to some running-position, the resistances on the board (243) are gradually brought into the circuit, the field current in c is weakened, and the motor (240) therefore accelerates until it attains a condition of equilibrium corresponding to the new field resistance. Moving back the radial arm does not cause the closing of the switch (269), as this is held open by a magnet. The magnet has sufficient strength to hold the switch open, but not sufficient to move it from its closed position without the help of the radial arm.

From the foregoing it will be seen that the motor and pump can be made to run at any speed within the assigned limits, and that the motor cannot start with any but its maximum field. To obtain the same result by automatic means, a link (252) is fixed in the left-hand position, as shown in full lines. This link may be regarded as a positive pole, of which the corresponding negative pole is the armature connection (a) of the main motor. In passing from the positive to the negative pole any current must traverse the armature of a small generator (257), and will therefore be influenced by any potential which may be produced in this small machine, which is worked by gearing off the main motor and partakes of all its changes of motion.

It will be observed also that between the two poles above mentioned there are two other circuits in parallel with the field coils of the small generator (257). The object of each of these circuits is the same, namely, so to reduce the field current in that generator that the potential it produces shall be negligible in its influence upon the field of the main motor. Therefore, when either of these circuits is closed, as is arranged shall be the case at starting, the main motor (240) must run with its full field and at its minimum speed.

One of these circuits passes through a switch (259), and the first operation of the starting-panel is to close this switch, which is not opened again until the motor has attained its first running-speed. The motor can now run safely at its lowest speed, but provision has still to be made against its suddenly passing from the lower to a higher speed; this is prevented by the second of the above-mentioned shunt circuits, the essential part of which consists of a mercury column contained in a tube formed of perforated steel disks separated by insulating washers and clamped together as already described for the smaller units. When this tube is full of mercury, the current passes without resistance. This mercury column is in parallel with the field coils of the small generator (257), and it therefore takes the greater part of the shunt current,

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and again the potential of that generator is kept at its minimum value.

In order to provide for variations in the field strength of the small generator (257) and the consequent variations in the speed of the motor (240), the following further arrangements are made. The insulated disks forming the mercury tube, ninety in number, are connected to a set of ninety terminal studs opposed to another set of sixty studs forming the terminals of a resistance board (271). By suitably connecting the studs of these two sets of terminals the individual disks forming the U-tube can be given any desired value in terms of field resistance.

The U-tube is connected to the venturi tube of the 24-inch main in the manner described for the other pumping units, so that when the venturi head is a minimum both legs of the U-tube are filled with mercury; all the disks are then in contact with it, and consequently all the resistance units of the board (271) are short-circuited. The field of the small generator (257) must, therefore, be at a minimum, giving the minimum speed in the main motor; and, as this position of the mercury corresponds also to the condition required for working the starting-switch (57), it follows that the motor can start only with its strongest field.

When the velocity of flow in the distributing-main increases, the pressure in the venturi throat decreases correspondingly, and the mercury is drawn out of the right-hand limb of the U-tube into the mercury reservoir (b of 242). The result is that certain disks in the upper part of the tube (241) are deprived of their mercury contact, and the only path for any electric current passing through the circuit is through such of the resistance elements as are connected to the disks thus left exposed. There is, therefore, a reduced current in the shunt and a correspondingly increased current in the field coils of the small generator (257). The increased potential due to this holds back some of the current in the field of the main motor, and, by weakening it, leads to an increase of speed.

This increase can be made to proceed at any desired rate, as the mercury falls in the U-tube, by properly adjusting the amount of resistance introduced as the surface of the mercury passes each of the disks of the U-tube; and any desired ratio between venturi-head and pump-speed can thus be obtained.

Occasionally it is necessary to stop and restart the machinery at a time when the flow through the main is such that the mercury column is in a position other than zero. In such a case the machinery could not safely be started, as the motor would at once pass to one of its higher speeds. To provide for

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such a case a disk and radial arm (271), similar to the control (243) of the shunt field circuit of the pump-motor, are introduced. On this the switch (274) closes by a spring as soon as the machinery stops, thus cutting out the resistances, and ensuring only the maximum field on restarting. The resistances must then be introduced by hand, by turning back the radial arm.

It will be observed from the foregoing that the introduction of the switch (274) impairs the absolutely automatic nature of the apparatus; for, although the machinery can be made to start, it will not increase its speed until the switch (274) has been opened by hand. This inconvenience can be remedied by the introduction of another switch (272), which cuts out 274 during ordinary automatic working. But it then becomes necessary to provide against another contingency, namely, leakage of mercury from the U-tube during the night. If this should occur accidentally, the motor would run rapidly up to its full speed, with considerable risk of damage; therefore a switch (275) is introduced, consisting of a mercury balance switch similar to 57, the cylinder being kept full by a branch on the U-tube. Should the latter leak, the mercury will run out of this cylinder, and the lever, being no longer balanced by its weight, will tip and open the contact; thus, in case of a leak of mercury the motor cannot start at all, and no boosting can be effected until the defect is remedied.

Up to the present purely automatic working has not been introduced, and the components numbered 272 and 275 in Fig. 6 do not exist, as an attendant always operates the switch 274 when the time comes to accelerate the speed of the pumps; but if at some future time the station shows itself so reliable that it can be left entirely alone, all the contingencies have been foreseen and appliances worked out to make it purely automatic and, it is believed, "fool-proof."

The pump for the large unit is of the same type as those for the smaller units, but has a 14-inch suction and 12-inch delivery. When tested its maximum efficiency, namely, 80 per cent., was at 350 to 450 r.p.m., and at a discharge ranging between 2,500 and 3,500 gallons per minute against heads of 30 to 50 feet; but it was capable of delivering 4,000 gallons per minute against 150 feet head when run at 750 r.p.m., under which conditions its efficiency was 76 per cent. As in the former case, the maximum efficiency corresponds closely with the mean work it has to do.

Conclusion.—The first three units have now been at work for three full working-seasons. During this time they have given perfect satisfaction; no defects of any sort have occurred, and

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the only ground for criticism is, as already stated, the rather poor efficiency which is always characteristic of this class of machinery.

Of the mercury rectifiers it is not yet possible to speak with equal confidence, as they have as yet only completed their first working-season; but the work which has been already carried out points very hopefully to complete success. Of their superior efficiency there can be no doubt, and maintenance does not seem likely to be a heavy item. The only trouble experienced so far is that there has been irregular action in some of the bulbs. On account of their large size the makers, for reasons of technical convenience, have provided each bulb with two anodes placed in parallel. In some of the bulbs there is a tendency for one anode to take the current in preference to the other, but this unequal distribution of current does not lead to any serious diminution in the total capacity of the bulb, as each of the two anode terminals has a large factor of safety. Only certain bulbs are subject to this defect, the cause of which has not yet been discovered. Some have broken in transit, and two appear to have lost their vacuum. When these imperfect bulbs are eliminated, there will probably be nothing to complain of.

There is a certain speculative element in a mercury rectifier. The bulbs are either good or bad. If the former, they are good for a very long time. If the latter, their badness is soon apparent and they must be renewed. The cost of renewal appears to be the only serious item in the expense of upkeep.

Were the Author called upon to design another boosting-station, he would, in the present state of his knowledge, certainly adopt mercury rectifiers, but he would make a strong effort to combine with them the method of speed-control by change of armature-voltage adopted in the earlier units, which is much simpler and more economical.

It appears certain that the boosting-station as a whole has fully justified itself. At the time it was designed it was the only practical solution of a grave economic difficulty produced by the high cost of engineering material; but although planned originally in a rather modest manner, its great practical value quickly led to its development on a much larger scale.

Although the fall of prices has now profoundly affected the economic aspect of the matter, yet it appears certain that, if the same problem had to be solved to-day, the solution would be the same. The effect of the boosting-station has been to double the capacity of the existing mains, at a cost which, including the capitalization of the value of the electric energy, is probably not more than half that of supplementing those mains by new piping.

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The true economic effect to the community as a whole is, however, greater than this. The load upon the electricity works is very favourable, consisting of a daylight peak; and the revenue derived from the sale of the electricity is probably highly profitable. This additional profit reverts to the community, since the electricity works are public property, and the inhabitants of the city are thus having returned to them a part of the charge for the water they consume. If the water and electric supplies were both municipal undertakings, as is so often the case in England, the economic advantage would be more apparent.

Although the Author is convinced that in the given instance the use of boosting-pumps was the best method of overcoming the inconveniences which were beginning to present themselves, he does not wish it to be supposed that he brings that method forward as being universally applicable. Each case must be considered by itself, and the conditions which point to the adoption of boosting are mainly the existence of a highly variable rate of consumption of water, a long and expensive pipe-line, and access to a fairly cheap supply of electricity. Where these conditions prevail, the Author believes that boosting will be found to be the cheapest method of meeting the demand.

The total cost to the 30th November, 1924, was £62,373. This includes all expenses for management and supervision, and therefore comprises all that has been necessary to bring the station into effective action. The bare cost of the machinery in England was £19,000. It is interesting to note that the cost per unit of power for the motor-generator sets was almost exactly double that of the rectifier set. The difference between £62,373 and £19,000 is due partly to freight, transport, and erection, but chiefly to the costs of the building, the main conductors, various pipes and valves outside the boosting-station, and experimenting and management during the first two years of working.

In conclusion, the Author wishes to thank the Chairman and Directors of the Monte Video Waterworks Company for their kind permission to make use of the information contained in this Paper, and to record his appreciation of the high value of the assistance afforded by Mr. G. H. Clapham, the engineer in charge of the erection of the station, to whose ingenuity many of the details are due, especially that very valuable adjunct, the mercury balance switch.

The Paper is accompanied by eight tracings, from which Plate 5 and the Figures in the text have been prepared.

[Addendum.

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ADDENDUM (4 Dec. 1925).

SINCE writing the Paper the Author has received figures which give the cost of working the boosting-station for one complete year, and also a carefully prepared estimate of the cost of laying a new main pipe having a capacity comparable with the increased flow in the existing piping due to the action of the station.

The cost of the boosting-station, added to the capitalized value of one year's working, amounts to £135,000, while the cost of the additional main pipe, including all the ramifications necessary to carry the water to all parts of the town, is estimated to be £325,000. The cost of obtaining the result by boosting is therefore only 40 per cent. of that which would be involved by the necessary extension of the piping.

These figures may be taken as applying to a general case, comparable with the one described, but in the case under discussion the comparison is still more favourable, as boosting for the supply of certain local points could not possibly have been avoided.

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Discussion.

Mr. Roger T. Smith congratulated the Author on his share of the courage required to introduce, first, a completely electrically-worked and controlled pumping-plant, and secondly, the mercury-arc rectifier instead of the motor generator, in order to increase the efficiency of that working. He expressed his appreciation of the extreme clarity of the Author's description of what was rather a complicated form of electrical control. A risk had certainly been taken in choosing the principle of working and controlling the booster pumps electrically, which was a new application; but no risk whatever had been taken in working out and providing for the safeguards, and for what might be called the calibrating apparatus which was provided to ensure success when the working was made fully automatic. Was the station now worked entirely automatically? The Author's claim that the apparatus was "fool-proof" was probably justified, and the happy result of the work was shown by the fact that 60 per cent. of the cost of the pumping-main, which would otherwise have been required in order to do the same work, had been saved by the boosting-apparatus. Generally, the Paper concerned the use of electrically-driven centrifugal pumps, which, he thought, had always been a success wherever they had been used. They had been especially successful when used as multi-stage turbine pumps for the supply of hydraulic power at 750 or 800 lbs. per square inch pressure in docks for working hydraulic apparatus. The latest of those installations had just been started in Cardiff docks, and was running very successfully. The booster pumps at Monte Video worked, of necessity, on the rising part of the pressure—quantity pump-curve, always a difficult thing to do, and wanting careful regulation. What was the maximum velocity of water in the pipes beyond the boosters? The Author spoke (p. 134) of making the slope of the quantity—head curve of the pump steeper. Could that be done simply by altering the design of the impellers, or were further changes necessary? Also, was the added head entirely absorbed in the increased friction in the pipe, or would the head of the supply be increased? The Author referred (p. 132) to numerous mechanical and hydraulic methods tried for obtaining the desired control; had he considered an electrically-driven hydraulic variable-speed gear for the working of Mr. Smith.

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Mr. Smith. the pumps? For instance, in such a gear as that invented by Dr. H. S. Hele-Shaw, M.Inst. C.E., the venturi pressure-tube, either working direct or through a separate relay, would only have to move the floating eccentric ring of the pump one way or the other. Also, the 6,000-volt supply could have been applied direct to the motor and thus save transformer losses. He was not suggesting that the way in which the matter had been worked out was not the best, but he was interested to know if the simple method of using the infinitely variable speed gear had been considered, and, if so, why it had been rejected.

Mr. Lupton. Mr. H. R. LUPTON remarked that two points presented themselves to him, as a pump-maker, for discussion. One was the pump regulation. The Author stated that the pumps themselves were standard. When pumping through a long main, in which the head was almost entirely due to friction, the pumping head was proportional to the square of the quantity. That, he thought, was very nearly an ideal characteristic for a centrifugal pump to work against, because the ratio of the quantity to the square root of the head was constant, and the variation in both quantity and head was entirely done, in the instance under discussion, by speed. Therefore the pumps were working under almost ideal conditions for retaining maximum efficiency. He was rather surprised at Mr. Smith's claim that they were working on the rising part of the characteristic, for that very reason. Mr. Lupton thought they were working on the successive peaks of the efficiency curves for the different speeds. That was done at the expense of a certain efficiency in the electrical equipment. But if those pumps had not been absolutely standard, but had been made with nearly radial vanes, so that the head—quantity characteristic did not fall, but rose slightly, which it could be made to do, would not some of the regulation which had actually been done, perfectly successfully, by electrical means, have been possible by hydraulic means? He was rather diffident about mentioning his second point, since it concerned an electrical problem, and he was not an electrician. He thought the motor characteristic itself could have been used perhaps slightly more than it had been for regulation, independently altogether of venturi meters. In the case of plants B, C, and D, he gathered that the motors were shunt wound, but with plant A the motor was of a differentially-wound compound type which had, of course, a slightly rising characteristic with increasing torque. He had attempted with very rough experimental apparatus to see what sort of regulation could be got on the motor alone. He had put some series coils on a shunt-wound motor and had made it drive an ordinary pump. He had increased

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the resistance in the shunt circuit until the latter was in reasonable proportion to the series windings which were opposing the shunt windings. Supposing the motor could be wound so that its torque was proportional to the square of the speed, the torques of both pump and motor would correspond to one another, and a considerable range of speed might be obtained before one overbalanced the other. Thus it could be anticipated that wide variation in delivery head would be obtained before torque balance would be re-established, even after a comparatively small initial variation in pump torque. The experimental plant ran quite steadily until the series ampere-turns rose to a certain proportion, which calculation would indicate to be about one-quarter of the shunt ampere-turns. After that point the plant became rather unstable. The speed characteristic rose rapidly, as was required, but a good deal of surging and sparking occurred. In spite of all that, however, a variation was obtained by merely opening the pump-delivery throttle-valve. While the delivery increased from 45 to 80 gallons per minute, the speed rose from 1,750 to 1,800 r.p.m., and the head from 50 to 55 feet. He did not claim that the results were good; the efficiencies were very poor indeed; but he thought it might be possible, by working somewhere near that critical value of the ratio of the shunt to the series coils, to get a fair variation. He did not claim that it could possibly be sufficient for the case which the Author had put, but it might easily be sufficient in other cases, especially where the pump was delivering against a static head instead of delivering downhill. He asked whether the Author had any figures showing how much regulation might have been obtained by the motors themselves being contra-compound wound without the help of any venturi-meter control at all. Mr. Lupton.

Mr. A. C. ANDERSON was struck by the extraordinary ingenuity with which the Author had tackled the problem; but he doubted whether it had really been necessary to put in so much automatic apparatus. It seemed to him that the saving which had been obtained by using boosting-apparatus might possibly have been rendered even greater had simpler plant been employed. It appeared that attendants were always in the pumping-station, and it was mentioned that the hand appliances had been of great service in making the various adjustments. The same point of view was also emphasized on p. 142, where it was stated: "the former method [by hand] is of considerable importance, as it is relied upon to replace the automatic method." Apparently, also, the largest pump was not yet operated automatically. From the Author's description (p. 144) he gathered that the mercury tube was complicated, and Mr. Anderson.

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Mr. Anderson. reference was made to leakage of the mercury; it was stated also that in case of such a leak the motor could not start, and that no boosting could be effected until the defect was remedied. How often had that leakage occurred? Apparently it had happened fairly frequently during the night. In connection with the larger pump, it was said that the attendant always operated a switch when the time came to accelerate the speed of the pumps. No doubt the Author had considered all those matters, but it seemed that ordinary rotary converters, over-excited so as to correct the power-factor, and combined with hand-controlled variable-speed direct-current motors coupled to ordinary pumps, would have been the best equipment. He was not quite satisfied as to why the station had been designed with the aim of making it entirely automatic. He imagined that a good deal of expensive experimental work had had to be done. The rotary converter, too, he thought, had distinct advantages over the mercury rectifier. It was mentioned that the only trouble which had been experienced so far was an irregular action in some of the bulbs. He thought perhaps a more important point was that the bulbs were delicate. Some of them had broken in transit, and two appeared to have lost their vacuum. He did not wish to be accused of being an enemy to progress, but he thought that in some instances automatic apparatus, especially on a fairly large scale, could be much overdone.

Mr. Ackermann. Mr. A. S. E. ACKERMANN said he would like to congratulate, not only the Author, but also the Author's clients. It was not often engineers had clients who were so enterprising as to accept what their engineer recommended in so whole-hearted a manner as they had done in this case. The only point of criticism he had to make was that the centrifugal pumps delivered at right-angles into the main. He would have liked to see some method of overcoming the loss of head due to pumping into a main at right-angles, especially where the sizes of the two were so nearly alike. On p. 135 some slight information was given concerning the capacities of the mains. It was not often that precise information could be obtained of hydraulic experiments with pipes 18 and 24 inches in diameter, and 12 miles long; he therefore asked the Author to give the whole of the data he had obtained in that connection.

Sir John Snell. Sir JOHN SNELL remarked that some years ago he made an investigation into the corrosion of the water-pipes in Monte Video, which had been caused by leakages from the electric tramway system in that city. He thought, therefore, that the directors of the waterworks company were the more to be congratulated on their broad-mindedness, in view of the application of electricity which they

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had made in their boosting-station. There was nothing really to criticize in the Paper from the electrical engineering point of view. The selection of the apparatus, the method of control, and generally the application of the electrical plant to this method of boosting, seemed to him to call for nothing but congratulation. There was one matter which was of great interest to suppliers of water from the economic point of view. The Author showed that the capital saved by the method which he had adopted, as compared with the cost of supplying the otherwise necessary mains for the same conditions of water-supply, was £190,000. Having regard to the value of money in Monte Video, he supposed that might be taken as equivalent to about £16,000 a year. It would have helped if the Author could have given a balance-sheet of the annual working-cost, which he must now know, of the boosting-station, including the cost of energy, the depreciation on the boosting-station itself, the cost of repairs and maintenance, and any other overhead charges which might apply, and also if he could have shown in figures what was the net annual saving as compared with the saved capital charges upon the otherwise additional cost of the pipe system. Sir John Snell knew little about the economics of water-supply, but he presumed that with the growing population, and with the growing consumption of water per head of population which was now taking place, there must be a constant requirement upon waterworks engineers to increase their mains, especially their feeder mains; and it might be that the application of a boosting-station of the kind described that evening would save a very considerable capital outlay. If it did, and having regard to the hours during which the extra pressure of water would be required for industrial purposes, he suggested that, worked in conjunction with a general electricity supply, it would have a double effect: it would tend to improve the load-factor of the water-supply (and improvement of the load-factor in any class of business made for economy), and it would most emphatically tend to improve the load-factor of the electricity supply. On those grounds, if the Author could give the information for which he had just asked, it might have an effect on the policy of water-works engineers in this country. The Author was to be congratulated, as were also his directors, for their courage. He was glad to see that the results, after several years' working, had borne out completely the Author's prognostications.

Sir John Snell.

Sir Archibald Denny.

Sir Archibald Denny asked why the Author had included the capitalized value of the first year's working, because he presumed that during the first year the working-costs were the same as during the second and subsequent years in supplying the necessary extra

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Sir Archibald Denny. pressure to deliver water to the customers. He was glad to hear that the mercury rectifier was now applied to such large powers. He had a small one working a cinema projector in his own house, and found that its efficiency was very high. One of the services which the Author had rendered to The Institution was to bring prominently before its members the fact that there was now similar apparatus for such high power.

Mr. Sayer. Mr. C. E. SAYER remarked that on p. 129 the statement appeared: "It is estimated that at the present rate of increase the boosting-plant will have to be continuously in action in about 20 years; but long before that time boosting will have become unremunerative, and the capital expenditure on a new pipe-line will have become justified." Some balance-sheet would be helpful in connection with that remark. What exactly was meant by the statement "long before that time boosting will have become unremunerative"? He took it the Author meant that the boosting of the whole quantity of water passing would have become unremunerative, but that the boosting-station would still remain in its present form, or, as it then would have been extended, to continue in action in the same way as it was doing at present. The loss of head in the pipes, although they were found to be clear of any internal encrustation, was considerably more than had been estimated; and further remarks from the Author on that point would be desirable.

Mr. Hanssen. Mr. C. T. A. HANSSEN congratulated the Author on his courage in adopting an entirely new line of automatic control. It was an example to engineers not to be too timid in adopting the latest and most advanced appliances. He would like to know, with Mr. Ackermann, why the frictional resistances in a pipe which was not corroded, and which was in practically new condition, should be 50 per cent. higher than calculation would indicate. Any information which the Author could give on that point would be valuable to water-engineers, to whom the question of frictional resistance was of primary importance.

Mr. Thorpe. Mr. W. H. THORPE remarked that the basic principle of pressure boosting to accelerate the flow of water in pipes had characteristics common to most sound engineering propositions, namely, simplicity and—when once stated—obviousness. An important aspect of the scheme described concerned cases where it might be proposed to adopt such a pressure-boosting arrangement, but where apprehension might be felt as to the effect of failure of the electricity supply or of part of the apparatus. That was hardly a serious objection in such an instance as the present, where, without boosting, the supply met the mean demand of the city. If the apparatus

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broke down, or if the current failed, the supply would merely be limited to what it had previously been; and even if that occurred when there was a heavy demand for water, it would only mean a slight interference with the amenities of the city, and no danger to the public health. Mr. Thorpe.

Dr. W. L. Lowe-Brown was pleased to see that the Author had paid so much attention to a point which, personally, he considered very important, namely, the factor of cost, which was the determining factor in choosing between various means of augmenting the supply. There was always a temptation to adopt a conventional way of doing a thing, and, when the conventional way was just as cheap, it was nearly always the best; but there were cases, like the present, when it should be ruled out because of its cost. He admired the Author's courage in adopting the mercury rectifier. It was to the engineers who showed courage that progress was due, and he felt sure that many of the numerous examples of the rapid obsolescence of electrical plant were due to lack of courage on the part of the designers, who had simply gone on in a rut, being too timid to adopt new features, even after their advantages had been amply demonstrated by theory and experiment, which were generally so much more reliable in electrical engineering than in ordinary civil engineering. Dr. Lowe-Brown.

Mr. M. B. Buxton drew attention to an important point brought forward by the Author's remarks on p. 131. The Author had evidently taken great trouble to invite all the chief manufacturers in Great Britain to meet his requirements; and Mr. Buxton thought that was an example of patriotic spirit which younger engineers might well follow. Mr. Buxton.

The Author, in reply, stated that the only respects in which the full automatic properties of the installation had not yet been developed were the two mentioned in the Paper. In order to cover certain contingencies, a switch had been put in which, though it did not prevent the motors from starting, prevented them from speeding up. The pumps generally started early in the morning, before the attendant had arrived, and worked for an hour or so on their first increment of speed. All the contingencies provided against had not yet arisen, but it had been thought wise to take every precaution. Mr. Anderson was in error in supposing that the large pump was not automatic. Hand regulation was introduced to enable the machinery to be tested when the demand for water was not sufficient to call it into use, and also to provide a cheap, and in the circumstances a sufficiently efficient, stand-by for use if any of the automatic appliances should be out of repair. The Author

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The Author. It had also been of great service in the preliminary operations by which the controllers were regulated. The provision against the leakage of mercury was intended as a safeguard only. It had not been installed, because the mercury had never leaked; but the precaution was mentioned in the Paper to complete the description. The mercury tubes worked very well. There was a very slight electrolytic action in spite of the oil, but not enough to cause any harm if the tubes were cleaned out about once a year, which was easily done. The delicacy of the bulbs was not a matter of great importance. They called for careful handling, and the value of a broken bulb or two had to be added to the capital expenditure; but, when once installed, their durability was very considerable, and it did not appear that the cost of upkeep would be at all high. When providing for the increased head in the first three sets, it had been necessary slightly to modify the interior of the casings of the pumps, so as to accommodate the larger impellers. Several speakers had referred to the high friction in the pipes. The work had been designed on a conventional basis with regard to hydraulic friction, but subsequent tests had shown that the conditions were extraordinary, and that the pumping head would have to be increased from 100 to 150 feet to maintain merely the designed pressure of supply. He was trying to find out the cause of that. With a complicated system of piping, involving long distributing-mains and numerous branches, frictional conditions prevailed which were quite different from those indicated by ordinary experiments and the formulas deduced from them. Even if the cause of that friction were never found out, the figures given in the Paper would still be valuable. The Author sympathized with the desire to study the figures upon which the pipe-discharges were based, but the figures in their crude form would probably not be intelligible to anyone unfamiliar with the work in detail. He had, moreover, recently come into possession of other figures beyond those used in the preparation of the Paper, and he hoped before long to be able to submit a further communication to The Institution dealing specially with that point. The use of a mechanical variable-speed gear had been considered during the development of the design, and the reason for its rejection would be, perhaps, best explained by a brief recital of the history of that development. In the early stages both coal and oil had been considered as sources of power, but the expense was found to be far too great. Then it was suggested that an attempt should be made to burn coal or oil only when power was needed; but that had been found very unsatisfactory: day by day the conditions changed, and it was

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never known from one day to the next when power would be required; and therefore constant attendance would be necessary. In the end the difficulty was solved by the use of electricity obtained from the public supply, the Government taking the responsibility for its continuity. The next problem was to effect the required change of speed. The use of variable-speed alternating-current machinery had been proposed, but he had not been able to find, in the course of his inquiries, any firm which would undertake to make suitable machinery. The same remark applied to rotary converters—no firm had offered one capable of automatic control. The use of some mechanical change-speed gear had appeared more promising; but, when figures were gone into, questions of cost and efficiency made it less so than the other alternative of direct-current electric plant. Before adopting the mercury rectifier, he had made very careful inquiries. Their advantages were a combination of high efficiency and high power-factor. They were cheap to install, economical to work, and, up to now, had proved quite reliable. Regulation by the means suggested by Mr. Lupton had not been very much considered, because the range of head required was more than the pumps and motors alone could deal with: they had to be controlled by some external agent. Further, although there was a direct relationship between the delivery and the head of a centrifugal pump when it was working against friction alone, that was by no means the case when the pumping head was a combination of lift and friction. In the present instance the combination was peculiar, since there was a "negative lift" on the pumps. The conditions governing the need for increased head were somewhat complicated. First there was the pressure at the various points of delivery, depending upon the level of the ground and the height of the houses. That was constant, whether the demand for water was large or small, and could be obtained only in some parts by gravitation. In addition there was the head required for friction, which varied with the quantity of water demanded. The head to be created by the pumps constantly varied according to the equation $y = v^2 + x - z$, where v, x, and z varied independently. *Fig. 3* showed that in one main, taken as example, a delivery of 112,500 gallons per hour was possible without the pump working at all, and the existence of that initial flow upset the ratio of head to quantity which was usually characteristic of a centrifugal pump. To secure economy in working, the head produced had to be coerced into agreeing with that required by the quantity of water being taken, and it was that necessity which lay at the root of the need for automatic working. There were, moreover, four mains, each of

The Author.

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The Author. which might develop different requirements; and there was no guarantee that those requirements would remain constant. On the contrary there was a certainty that they would vary considerably as the area of the district supplied extended. It was necessary, therefore, to provide some highly elastic method, and that devised by the Author, of using two opposed rows of electric contacts, which could be linked together in an almost unlimited number of combinations, had proved in every respect satisfactory. The automatic control, which was viewed with some suspicion by Mr. Anderson, was necessary for two reasons. First, in order that the pumping should start when it was required, and not at some time varying with the vigilance of the attendants, and secondly, to secure that the pressure produced by the pumps should never be greater than was required by the friction in the pipes. If that had not been provided for, a careless attendant might cause considerable damage; that danger was automatically provided against. This answered Mr. Smith's point as to the pumps working on the rising part of the curve; they decelerated also on the falling part, and from the point of view of safety and "fool-proofness" the latter action was the more important. He agreed with Mr. Anderson that automatic equipment could be overdone, but that had not occurred in this installation, which was the outcome of careful calculation of costs and a good deal of thought. The purely automatic part of the apparatus was a very small portion indeed: whereas the indispensable part cost thousands of pounds, the part which made it automatic cost hundreds. Perhaps 10 per cent. of the cost was devoted to the entire elimination of personal attendance. The principal working-expense was the current. It had to be borne in mind that in Uruguay electricity was not particularly cheap, even for the purpose under discussion, since it cost 2*d.* per unit. Therefore the method would show to much more advantage where electricity was cheaper. The capital sum of £65,000 represented the total expenditure of all kinds upon the boosting-station up to the time the Paper was written, and therefore included maintenance and attendance during the first 3 years of the station's existence, during which, as construction work had been continually in progress, the separate items could not very well be kept apart. The other sum, which happened also to be £65,000, was the capitalization at 5 per cent. of the cost of 1 year's consumption of electricity, based upon the experience of the first full year of working. Thus the saving of £16,000 per annum which Sir John Snell had calculated was net saving after the cost of energy was allowed for. The only sums which it might be necessary to deduct from that in the future were

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the costs of the special attendance and of repairs to the machinery. Those appeared likely to be very low. The present staff consisted of one electrical engineer and two mechanics, whose wages aggregated about £800 per annum; and they had so far executed without further cost all the repairs, and had spent besides considerable time on operations in connection with the waterworks as a whole, in the workshop attached to the boosting-station. He sympathized with Mr. Ackermann's objection to the right-angled junctions; but, since the velocity through those pipes never reached 3 feet per second, the head lost in turning a right angle was only a small fraction of a foot. The justification for the right-angle turn was that it was not quite certain which way the water would be required to turn at any particular moment. Sir John Snell had alluded to electrolysis, and the Author remembered the time, 20 years ago, when that was a very serious matter in Monte Video. Special attention was paid to the problem, and for years past he had never heard of any further trouble. Sir John Snell had also referred to the importance of the use of electricity from the point of view of improving the load-factor of the electrical station. The Author believed this to be so great an advantage that all the boosting could actually be done for next to nothing. With regard to Sir Archibald Denny's question why the first year's working expenses only were taken, perhaps it would be better if the text read, "the first complete working year." Mr. Sayer had raised an important question regarding the limit of economy of the boosting process, which could only be answered by carefully watching the development of the city's demands in future years and the true cost of electric energy when the improved load-factor was taken into account. The dominant factor was always the rate of growth of the demand for water. As that continued, so did the expenditure upon electricity continue to increase, and *Figs.* 2, 3, and 4 showed that the rate of increase of power was very much higher than that of the demand for water, so that the greater the demand, the higher was the cost of pumping per unit. If provision were made for the extra quantity by increase of piping, the reverse condition prevailed, namely, the larger the quantity, the less the cost per unit; it followed, then, that a point was reached when the cost of boosting exceeded that for interest upon piping, and that showed the limit at which boosting was remunerative. In the case under discussion, however, a partial halt might have to be called before that point was reached, as it was highly probable that, by laying selected portions of the £325,000 worth of piping, the cost of boosting might at an early date be reduced in a degree more than proportionate to the partial

The Author

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The Author. expenditure of capital. He thanked Mr. Thorpe for accentuating a difference which was often neglected by engineers, but which was of real importance, namely, that between an inconvenience and a disaster. Failure of the electric supply would involve the town in the first, but not the second of those experiences, a point which it was important to bear in mind in judging the boosting scheme. He wished to supplement in two respects what was stated in the Paper concerning the electric control. He had hardly done justice to the balance switch. This could be made to work with much greater accuracy than that stated, as the lag of 2,000 gallons per hour (p. 134) was to a large extent deliberately introduced, being one of the devices by which quick make and break were effected. The description of the method of control of the large motor, given in the Paper, was based upon the method evolved by the Author, which the experiments of Mr. Lupton demonstrated to be theoretically sound; but the electrical experts to whom the design in detail was unreservedly handed over found it necessary to modify the details so as to bring the field current down to the limit imposed by the mercury tubes. The control was therefore effected as in an ordinary shunt-wound motor, with the exception that the energy in the field current was absorbed not by heating resistances but by causing the small motor to do in a varying degree a part of the work of the big one. Then only the greatly reduced field current corresponding to the small motor had to be dealt with by the mercury tubes and their attached resistances.

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Correspondence.

Mr. G. H. CLAPHAM considered, from what he had seen in actual operation of the station, that automatic regulation of the pumps was not only worth while, but indeed essential, because hand regulation was impossible. Experience showed that one attendant could only regulate one main approximately, by always giving a higher pressure than was needed, with consequent waste of power. To secure the greatest economy in working-costs, two conditions had to be fulfilled: a pumping-set must never start before it was absolutely necessary, and it must stop the moment that the gravity head would be sufficient for the main. Those conditions were fulfilled by the use of the automatic balance switch. After a pumping-set had been started at its lowest speed, there must be automatic regulation of the pump-speed, so that the pumping pressure added to the main at all times would be just sufficient to maintain a constant predetermined pressure at a selected distribution-point in the city. That condition was fulfilled by the adoption of a mercury regulator which governed the pump-speed over a range of about 400 revolutions per minute. The sensitiveness of that apparatus was much greater than that of the recording water-meters, as, while both were actuated from a mercury column in their respective U-tubes, the meter transmitted the motion of the mercury to the recording apparatus by a float rack and pinions, whereas the regulator U-tube controlled the speed of its pump-motor directly. If an attendant using hand regulation followed precisely the pen lines as shown by the meter, there would still be some error. It would have been necessary also to have four attendants, one to regulate each of the four mains. It was found that close regulation of the pumps to their work was very necessary, as it frequently happened that while a main was being boosted, sudden and quite unexpected drops in the consumption occurred for a few minutes. If during that time there were no regulation on the main, the pressure in the city might rise to a dangerous amount. With a regulator in use, the pump-speed and the pressure dropped with the consumption. While the duty of a regulator was to increase the pump-speed and the pressure by degrees as required, until the peak of the day was reached, it was equally essential that the speed and pressure should be reduced in the same way, until the lowest motor pump-speed was reached, Mr. Clapham.

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Mr. Clapham. when the balance switch stopped the motor, and gravity head took charge of the main until the pump was required again. In actual operation of the station the four pumping-sets started at very different hours on different days. While in very hot weather all might start between 6 and 6.30 a.m., in winter or on rainy days the start would usually be some hours later; in some cases only one or two pumping-sets might start during the entire day, and perhaps run each for 1 or 2 hours, against about 11 hours each for a normal hot day. Such conditions were met by the balance switch operating only when necessary: if it were not for that control the attendants would undoubtedly have run the pumps more than was necessary. As the station was worked at first during the experimental stage by hand, Mr. Clapham had had some difficulty in restraining them, but they became convinced later when the balance switch was in control, as there were never any complaints as to lack of pressure. Although ninety contacts were provided on the largest of the regulators for experimental purposes, it was found in all cases that forty were ample, so that any complications from that source need not be taken into consideration, especially as the connections, when once made, remained the same for long periods.

Mr. Hammick. Mr. H. A. HAMMICK observed that most water-supply engineers had to face the problem, the solution of which at Monte Video was described by the Author. It would be interesting if the Author would give further information as to the local conditions. Was the water-pressure in the town bound by law to exceed a certain minimum, or was a certain pressure arbitrarily taken as a reasonable minimum? If so, what was that pressure? What was the cost of electric energy per unit? It would appear that apart from the supply to the remainder of the town, boosting for the high parts around "F" (*Fig. 5*, p. 129) would have been necessary. But except for this boosting, larger pipe-lines would suffice to maintain pressure. The cost of larger pipes was stated to be about two and a half times the cost of the boosting-station as constructed. However, as time passed, the velocity in the existing pipes would increase until the additional losses in friction would absorb the power available from the pumps. Then, apparently, larger mains would be provided. For further increased consumption, would the existing pumps be again used for boosting, or would their capacities be then too low? What would be the velocity of flow in the existing mains when the frictional loss became equal to the maximum boosting-capacity of the pumps? The figures for cost showed that, for the present, boosting was cheaper than additional mains. But it was

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anticipated that in future additional mains would be required. Was not the present solution a temporary palliative rather than a cure? It might well be that a low present cost was financially more convenient than a high present cost; but finally would not the total cost of the present scheme of first boosting, and then adding mains be higher than it would be if additional mains had been added with only sufficient boosting to provide for the high parts of the town? The life of an electrical boosting-plant would probably be much less than that of pipes, unless local conditions were very unfavourable to pipes, so that the sinking-fund for the electrical plant must be calculated at a higher percentage per annum than that for pipes. The Paper was especially instructive and interesting to those who were connected with the water-supplies of towns in the tropics having very large differences between maximum, minimum, and average daily consumptions. Apparently in Monte Video the maximum hourly rate of flow for the whole year had been provided for. That would appear, to one accustomed to the penurious East, to be extremely generous. Mr. Hammick.

Mr. S. M. HOLDEN considered that the whole installation appeared to have been very well planned to reduce operating-charges to a minimum. It seemed to him, however, in connection with the lay-out of the main water-distributing pipes, that the best position for the pump delivery branch would have been directly in line with the venturi main. The straight length of piping before the venturi meter did not appear to be long enough to destroy all eddies before metering. Mr. Holden.

Mr. F. W. MACAULAY congratulated the Author upon his valuable Paper, and upon the successful manner in which the late Mr. Parsons and he had solved a difficult problem. He had read the Paper with much interest, especially as the question of "boosting" had been very much before him in recent years in connection with the water-supply of Birmingham. As most waterworks engineers were aware, the Elan aqueduct, which conveyed the water from the Welsh gathering-ground to the district of supply, was about 73½ miles in length, and consisted, for half of that length, of siphons, on which had been laid, originally, two parallel mains about 42 inches in diameter. There were eleven siphons on the aqueduct, three of which were about 17, 9, and 4½ miles in length respectively, the remainder being quite short. A few years ago, when necessity for increasing the capacity of those siphons became imminent, a suggestion was made to the Water Committee from an outside and somewhat authoritative source that, instead of laying a third main Mr. Macaulay.

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Mr. Macaulay. for this purpose, the proper policy to adopt would be to install boosting-pumps on the mains of the three longest siphons. The suggestion was put forward in considerable detail, into which, however, it was not necessary to enter here. Mr. Macaulay reported against the proposal, but, in view of the manner in which it had been put forward, requested his Committee to obtain an independent opinion. Messrs. Willcox and Raikes, MM. Inst. C.E., were thereupon instructed to report on the proposal. They supported Mr. Macaulay's contention that the proposition was not economical. The boosting stations were to be, of necessity, in very out-of-the-way situations, far from any source of energy, and with the worst possible facilities for fuel-supply. The load to be dealt with would be fairly steady, increasing it was true, but very gradually. Apart from minor objections, it was shown very clearly that interest and sinking-fund charges plus working-costs for a third main, complete throughout, would work out more economically than the smaller capital charges plus much greater and more uncertain working-costs under the boosting scheme. On this the Water Committee decided to lay a third main. Before, however, any of this work was put in hand, the cross-over scheme [1] for the third main came along and weighted the scales very heavily against mechanical boosting on the original siphon mains. The foregoing illustration emphasized the Author's statement that "each case must be considered by itself, and the conditions which point to the adoption of boosting are mainly the existence of a highly variable rate of consumption of water, a long and expensive pipe-line, and access to a fairly cheap supply of electricity." Mr. Macaulay was inclined to add a warning against lightly substituting boosting by mechanical agency, with a fallible source of energy, for the cheapest and most reliable agent available, namely, gravitation, unless the economic advantage to be gained by so doing was really great.

Mr. Mountfort. Mr. L. F. MOUNTFORT observed that pressure-boosting as an alternative to the duplication of supply mains had for some time been recognized as providing a promising field for investigation, especially for long pipe-lines with rather flat gradients. The chief desideratum was the availability of a cheap supply of energy which could be turned on or off at a moment's notice, since the boosting-plant, when first installed, generally operated at a poor load-factor and, in fact, showed to most advantage when compared with the cost of a new pipe-line for the very reason that

[1] Minutes of Proceedings Inst. C.E., vol. ccxi (1920), p. 30.

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the load-factor was low. Under these circumstances the most suitable plant to install consisted of electrically-driven centrifugal pumps using current purchased from a large electrical undertaking. The problem, reduced to its simplest terms, was that of providing a suitable automatic control of centrifugal pumps to give a discharge which should increase with an increasing delivery-head. It had sometimes been suggested that to meet these or similar conditions it would be possible with a suitable design of pump, together with a driving-motor with a rising speed–torque characteristic, to obtain a head–quantity curve following approximately the requirements of the pipe-line, and thus to make the operation entirely automatic and dependent upon the inherent regulation of the pump, without the necessity of any special controlling apparatus. Mr. Mountfort had had occasion, about 15 years ago, to investigate the possibilities of a similar arrangement in connection with a large sewage pumping-station where the problem, however, was the simpler one of obtaining a variable pump-discharge against a constant head. Although the conclusion was reached that such an arrangement was impracticable and had serious limitations, yet the point was of some theoretical interest, and he proposed to consider its application to the requirements of a pressure-boosting station. To approach, even theoretically, the desired conditions would necessitate the employment of a pump with radial or forward-curved vanes operating on the rising part of the head–quantity curve, which could be further steepened by employing a driving-motor of the differentially-compounded continuous-current type. Such a motor was, however, rather unstable, and it would not be practicable to design it to give more than about 15-per-cent. rise in speed from no load to full load without dangerously reducing its margin of overload capacity. A pump with radial vanes, driven by such a motor and arranged to operate at the top of its characteristic at maximum load (which, with this type of pump, was theoretically its maximum efficiency point) would not give a drop of more than 20 per cent. of its maximum head between full and half quantity, and even then would show a tendency to surge. Mr. Mountfort

As the pumping-plant had to operate over a range of half to full quantity against a head varying from zero to maximum, it was evident that the above combination could meet the requirements of a variable discharge only by imposing great variation of head upon the distributing system and operating at a considerably reduced efficiency. Moreover, even if a combination of motor and pump could be designed to give an inherent regulation with regard to head, approximating to that required on the supply main with a reasonable

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Mr. Mountfort. efficiency, the phenomenon producing variation in discharge could only be the fluctuating head in the pipe due to variation of demand, and the arrangement would have no stable working point whatever. The only satisfactory method of control was the one adopted by the Author, namely, to vary the pump-speed by a separate control depending for its operation upon the variation in *quantity* passing into the pipe-line, and Mr. Mountfort considered that the Author was greatly to be congratulated upon the ingenuity displayed in the design of the speed-controlling mechanism and its successful application to the special requirements of the plant. He would be glad if the Author could give more information regarding the economics of pressure-boosting systems. It was stated (p. 129) that the expenditure on a new pipe-line would be justified long before the boosting-station had to operate continuously. The operating load-factor of the station appeared from the curves given to be about 30 per cent., which would apparently be increased to about 50 per cent. for full-time operation; and as the increase would affect only the operating-costs of the station, it was difficult to reconcile that statement with the figures of cost given in the Paper and addendum. He would suggest that the upper limit of capacity for a boosting-plant was governed largely by the question of pump-efficiency. With a station designed to double an existing supply, it would be found that the requirements necessitated working over a range of about 10 to 15 per cent. above and below the maximum-efficiency point of the pump characteristic, which was quite within a reasonably efficient working-range; with every increase in the relative capacities of boosting-plant and existing pipe-line the range must become wider, with a consequent drop in average efficiency. It was probable that the extreme upper limit, on purely financial grounds and under circumstances favourable to the boosting-plant, would be a capacity equal to three times that of the existing main, but considerations of pressure and velocity in the main would probably reduce that to a maximum capacity not greatly exceeding double the existing supply. This capacity would also probably be the lower limit in the case of a single existing supply main, as a duplicate main of similar size would be the natural alternative for extension. It would seem, then, that a boosting-plant to give 100 per cent. increase in an existing supply possessed features of peculiar interest, and its possible economy was worth consideration. The characteristics of the existing supply-system which determined the economical possibility of the pressure-boosting plant were the maximum velocity in, and the diameter of, the existing main, and the load-factor of the demand.

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He did not quite agree with the Author in his statement (p. 147) that length of pipe-line was in itself a determining factor, as it must enter in an equal degree into the cost of both alternatives. The most sensitive criterion by which to judge the possibilities of a pressure-boosting station was the maximum velocity in the existing main when delivering its full quantity under existing conditions. It would appear from calculation that a maximum velocity of 3 feet per second was on the dividing line, above which it would probably not pay to install a pressure-boosting plant and below which its possibilities were worth investigating. It would probably not pay to install any boosting-station which would impose a maximum velocity greater than 6 feet per second upon the supply main; but he would be glad to have the Author's opinion upon that matter. A pressure-boosting station such as that described in the Paper had the advantage that it enabled the cost of an additional supply main to be deferred until the time when it could be immediately used to its full capacity, and the station could then be extended to meet the excess demands until the growth of the system necessitated the installation of a third main. The economy of the boosting-plant as compared with a new main would show to greater and greater advantage with each subsequent main added to the system. He congratulated the Author upon having presented a Paper containing such a mass of engineering detail described in so lucid a manner. Mr. Mountfort.

Mr. F. C. TEMPLE remarked that the information contained in the Paper was satisfactory to those responsible for the waterworks in Jamshedpur, where provision had been made for a future boosting-station. When the new water-supply system was installed, 3 years ago, it was most convenient to deliver the water into the town by separate mains working under different pressures. One of those mains, an 18-inch concrete pipe, led to the base of a water-tower containing two tanks, one at ground-level and one with its draw-off 35 feet, and its top water 55 feet, above ground-level, and the main also continued to another reservoir 1 mile away. The pressures required to deliver the necessary quantity of water into the top reservoir and the distant reservoir were the same. Pumps were already available capable of fulfilling the highest duty while everything was new and in good order. Instead of providing separate pumps for the different mains, it had been decided to use the existing pumps with additional similar pumps, all pumping into a large main somewhat like a bus-bar in a power-house, and to regulate the pressure on each new main. As a time was likely to come when the pumps would not be able to deliver the necessary Mr. Temple.

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Mr. Temple. quantity into the top reservoir and the distant reservoir, blank flanges had been left on the mains in the tower, and valves had been inserted and space left in the house for installing boosting-pumps to lift the water to the top reservoir and drive it to the distant reservoir. From the Paper it appeared that the designers were correct in their assumption that boosting would be cheaper than providing larger mains and pumps.

The Author. The AUTHOR, in reply, observed that the water-pressure in the town was that which had been found necessary in practice: a maximum of 80 feet above ground-level was aimed at in the thickly-populated parts (E on *Fig. 5*), where buildings were three or four storeys high; in the highest parts of the town (F on *Fig. 5*), which at present were suburban and had much lower buildings, the pressure maintained was 57 feet. One advantage of the boosting method was that these pressures could be regulated to a nicety, so that practical conditions could be fulfilled in one district without supplying water at an unnecessary pressure in another.

The estimates were based on an average cost of electricity of 1·85*d.* per kilowatt-hour. Point F was not the only one for which boosting was indispensable. At point E also the hydraulic level was equal to that of the reservoir D, so that no pipe, however large, would have been satisfactory at times of maximum demand. Besides this, it was in itself economical to boost the pressure, as he would show later.

Mr. Hammick's forecast was correct, in that, as time passed, more and more use had to be made of the boosting-pumps. The increased flow of water due to the ever-growing consumption caused them to work at a higher speed and so put more pressure on to the mains to neutralize the increased friction. This process must, however, have a limit, for two reasons: one was that the permissible pressure upon the pipes would at last be reached; and the other was that, as the cost of pumping increased approximately as the cube of the quantity pumped, the process became at length inordinately expensive. When either of these limits was attained, it was time to lay a new pipe. When the new pipe was brought into use, the work of the boosting-pumps adapted itself to the new conditions, but gradually increased as the quantity of water became great. Thus the combination of pumps and pipes always gave its best efficiency. The method was, therefore, not merely a palliative, as Mr. Hammick suggested, but a permanent solution of the problem under all the changes which were likely to occur for many years to come. Whether the present boosting-pumps would be sufficient

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must depend upon the size adopted for the future main. They would certainly be large enough for some time after its installation. The Author.

The question of the economical velocity of water in a pipe when that velocity was produced by pumping was an important one, which, so far as the Author's knowledge went, had never been adequately treated. Its determination involved several factors which were often not easily ascertainable, namely: diameter of the pipe, condition of the pipe, cost of the pipe, cost of power, and load-factor. Fortunately they were all known in the present case, and the Author had worked out the required solution. The cost per unit pumped was obtained by finding the sum of the cost of the pipe and of the power used and dividing it by the quantity of water delivered. By differentiation the condition could be found when this cost was a minimum. In the given case it was found that for the three diameters which had to be taken into consideration, namely, 18 inches and 24 inches for the existing mains, and 30 inches as the probable diameter of the next future main, the most economical discharges were :—

30-inch main,	14,150	gallons	per	minute	=	7·70	feet	per	second velocity
24-inch ,,	5,705	,,	,,	,,	=	4·85	,,	,,	,, ,,
18-inch ,,	1,824	,,	,,	,,	=	2·77	,,	,,	,, ,,

The equation giving these results, which was found from the differentiation of the known factors, was $Q = 0{\cdot}0167\ d^{4{\cdot}095}$. In the case under consideration the quantities which might be economically delivered were so large that the problem did not arise; before the economic limit was reached, the head would have arrived at the bursting limit of the pipes; but for pipes of less than 18 inches diameter overall economy would become the ruling feature.

It was true that the maximum demand had been the basis for all calculations. That was due to the peculiarities of the climate and to the habits of the inhabitants, which called for a large supply of water for cooling the paved yards of the houses during spells of excessively hot weather. A high consumption therefore occurred at very uncertain intervals.

Mr. Holden's criticism as to the position of the branches seemed at first sight to be justified, but the circumstances rendered a direct run less desirable than would appear. The water was supplied not only by the pump but also by the gravitation by-pass alongside it, and they could not both be in line with the venturi meters. In any case the loss of head due to the double acceleration was very small.

Mr. Macaulay's remarks illustrated what the Author had said

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The Author. about considering each case separately. He would certainly never recommend boosting in connection with the Elan aqueduct.

Mr. Mountfort mentioned a point which had not escaped the notice of the Author. He believed, however, that in view of the wide range over which control had to be exercised and the great economy produced by rendering such control completely effective, he had adopted the best method. No doubt other circumstances might be effectively met by a specially-designed centrifugal pump. The reason why he have given the length of the pipe as one of the factors determining the advantage of boosting was that a long pipe required more friction head and therefore gave more scope for the economies which boosting introduced. A more general statement would have been made by the substitution of "high friction" for "extreme length."

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12 January, 1926.

SIR WILLIAM HENRY ELLIS, G.B.E., D.Eng., President, in the Chair.

On the recommendation of the Council, the members present elected, by acclamation, as an

Honorary Member.

General the Right Honourable JAN CHRISTIAAN SMUTS, P.C., C.H.

The Council reported that they had recently transferred to the class of

Members.

MAHMOUD ALY, M.Eng. (*Liverpool*).
WILLIAM LOWE ATKINSON.
GEORGE ERNEST BENNETT, M.Sc. (*Manchester*).
ROBERT BRUCE.
THEOBALD STUART BUTLER.
GEORGE EDWIN BOHUN COULCHER.
GERALD CURRY.
GEORGE WILLIAM DODDS.
FREDERICK CHARLES GRIFFIN.
THOMAS REGINALD GRIGSON, O.B.E.
ALFRED JOHN HALL.
GILBERT JAMES HARTLEY.
GEORGE ERIC HOWORTH, *M.C.*, B.Sc. (*Manchester*).
RONALD CLAUDE YVONNE KIRKPATRICK, *M.C.*
WILLIAM EDWARD LLOYD, *M.C.*
WALTER MACLACHLAN, B.E. (*National*).
EDWARD LOVELL MANLEY.
ALGERNON EDWARD MANN, O.B.E.
ROBERT PEEL MEARS, B.A. (*Cantab.*).
JAMES EDMONSON MONK, B.Sc. (*Victoria*).
DAVID JOHN MORRIS.
ALLAN STEPHEN QUARTERMAINE, *M.C.*, B.Sc. (Eng.) (*Lond.*).
STANISLAUS GEORGE REILLY.
DOUGLAS HENRY REMFRY, B.Eng. (*Calcutta*).
LEONARD JOY SPILSBURY, *M.C.*, M.A. (*Cantab.*).
HUGH JOHN HARRY STEDMAN.
HENRY TRENCHAM.
THOMAS HAYNES UPTON, O.B.E., M.Sc., M.C.E. (*Melbourne*).
WILLIAM KELLY WALLACE.
OWEN THOMAS WOOD.

And had admitted as

Students.

WALTER JAMES ANDREWS.
RICHARD SPEIGHT BELLHOUSE.
ARTHUR THOMAS BLACK.
NORMAN PUMFRETT BRAND, B.Sc. (Eng.) (*Lond.*).
GEOFFREY CAREL ALLEN BRYAN.
ARTHUR JAMES BUTLIN.
HARRY ALEXANDER FRANCIS CANFIELD.
GEORGE HENRY CLEGG.
JOHN NOEL COCHRANE.
HERBERT CYRIL DAVEY.
STEPHEN PHILIP DE LÁSZLÓ.
HERBER HOWELL EVANS.
JOHN DOUGLAS FIDLER.
ROBERT LOUIS FITT.
ALISTAIR JOHN FRAZER, B.Sc. (Eng.) (*Lond.*).

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Students—continued.

HUGH JOHN VINCENT GLENDAY.
SHEIKH ABDUL HAKIM, B.A. (*Punjab*).
KENNETH MILLINGTON HARPER.
ROBERT MILROY HAYES.
KENNETH ALAN HAYNES.
CHARLES THOMAS HENFREY.
FRANK HEWSON.
HAROLD JAMES HILL.
GUY HODGE.
JOHN DENISON HOW.
ALFRED HARRY FRENCH JIGGINS.
JOHN OLDFIELD KERR.
WILLIAM FREDERICK LONGFIELD.
REGINALD MELVILLE WARD LOWE.
SEIFOLLAH MOSHAR OUTAN.
RONALD FLINT PEARSON.
HOWELL VIVIAN PHILLIPS.
WILLIAM JOHN RANKIN.
RICHARD SMITH RENDLE.
JAMES REGINALD SPENCER RHODES.
JAMES PEEL RICHARDS.
THOMAS LIDDELL ROBERTSON.
FREDERICK CLIFFORD SAGE.
JOHN ALEXANDER TEMPLETON SHANNON.
ERIC LANGHAM SIMPSON.
JACK MONCRIEFF SIMPSON.
ERIC FRANCIS SMITH.
FRANCIS ARTHUR SMITH.
LEONARD JAMES GRAVES SMITH.
PHILIP MARTIN SPENCER.
JOHN FREDERICK GORDON SPRING.
HOWARD JOHN SQUIRES.
GEORGE THOMAS STARLING.
GORDON WICKEDÉ RICHARDS STRICKLAND.
ERNEST EDWIN SWEETING.
HAROLD GUY PERCY TAYLOR.
NICHOLAS WHEAR.
MATTHEW TWEEDIE BAILLIE WHITSON, B.Sc. (*Edin.*).
CLIFFORD ISAAC WILKS.
ROGER JOHN WILLY.
BERNARD JOHN WITHERS, B.Sc. (Eng.) (*Lond.*).
FREDERICK TOM WOOD.
FRANK RUSSELL WRIGHT.

The Scrutineers reported that the following candidates had been duly elected as

Members.

LOUIS GEOFFREY BUCKNELL, B.E. (*Sydney*).
ARTHUR ELLIS.

Associate Members.

JOHN FRASER ANDERSON.
DAVID GLEN BLYTH.
DAVID HUGH CAMERON, B.Sc. (*Edin.*), (Stud. Inst. C.E.).
HAROLD LANGDON COPESTAKE, B.Sc. (*Bristol*).
STANLEY JOHN DAVIES.
JOHN LESLIE FEATHERSTONE FENWICK.
FRANCIS MATTHEW FULLER, B.Sc. (Eng.) (*Lond.*).
MORGAN BESLEY HATFIELD, B.Sc. (Eng.) (*Lond.*).
VICTOR RADCLYFFE HUSBAND, B.Sc. (Eng.) (*Lond.*).
REGINALD ALEXANDER KIDD, B.Sc. (*St. Andrews*) (Stud. Inst. C.E.).
LUCIEN BRUCE DOUGLAS KIRKPATRICK.
HENRY MONTAGU LYONS.
KENNETH SAMUEL MCCLARE.
CHRISTOPHER MARRIAGE MARSH, B.Sc. (Eng.) (*Lond.*).
ARTHUR LESLIE MAYLAN (Stud. Inst. C.E.).
COLSTON WILFRID JAMES MOUNTFORD, B.Sc. (Eng.) (*Lond.*).
EDWARD OSCAR PEPPER.
FRANK ALAN RAYFIELD (Stud. Inst. C.E.).
JOHN HARVEY MELVILLE RICHARDS (Stud. Inst. C.E.).
GEORGE HORACE TODD.
NICHOLAS TREWHELER.
PERCY JOHN WELLS (Stud. Inst. C.E.).

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The following Paper was presented for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

(*Paper No. 4573.*)

"High-Level Bridge, Newcastle-upon-Tyne: Underpinning and Repair of Foundations of River Piers."

By Cyril Francis Bengough, M. Inst. C.E.

The High-Level bridge, Newcastle-on-Tyne, was built under an Act obtained by the Newcastle and Darlington Junction Railway Company in 1844. Work commenced in 1846, and the bridge was opened without ceremony on the 14th August, 1849, being brought into general use in February, 1850. Until the opening of the King Edward bridge in 1906 it carried the whole of the railway traffic passing by the East Coast route through Newcastle-on-Tyne to Scotland. The engineers for the bridge were Robert Stephenson and Thomas Elliot Harrison, Past-Presidents Inst. C.E.

Description of the Bridge.

The bridge (Fig. 1, Plate 6) has six main spans of 125 feet, four of which are river spans with ashlar piers and bowstring girders composed of curved ribs of cast iron, the ends of the ribs being tied together by wrought-iron tie chains. The upper platform carrying the railway is supported by cast-iron columns carried on the ribs, and the roadway is suspended from the ribs by wrought-iron suspension rods, at a level of 22 feet 7 inches below the rails. The height from H.W.O.S.T. to the underside of the girders is 82 feet 10 inches.

Three of the piers stand in the river. At the time the bridge was built the depth at low water was 3 feet, but the present depth at low water is 27 to 30 feet, with a rise of tide of 15 feet. Four of the piers, one of which, the northernmost, is now enclosed by the quay frontage, were founded on memel timber piles. There are

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121 piles to each pier, spaced 4 feet apart centre to centre in both directions.

Crowns of 12-inch by 12-inch section are mortised and bolted to the piles transversely to the piers, and to these crowns are spiked two layers of 3-inch planking laid diagonally to form a platform for the masonry. The whole of the piles, for a depth of 5 feet below the top of the crowns, are surrounded by a bed of Roman cement concrete.

The following description of the formation of the river-bed is given in the notes appended to Captain B. M. Laffan's report, dated 11th August, 1849, of his inspection of the bridge on behalf of the Board of Trade.

"The Tyne at Newcastle is 520 feet wide with a depth of water not exceeding 3 feet at Low Tide. The River, however, flows over a bed of loose sand almost of the nature of a quick sand extending to a depth of 31 feet. Below this is a layer of coarse gravel 4 feet in depth resting upon 8 feet of stony yellow clay, and below this, and 46 feet from L.W., we at length reach a firm foundation in a bed of freestone."

The formation at the south pier, as proved by the excavation and borings, is as follows:—The strata rise quickly from north to south. On the north side of the pier the sand extends to a depth of 16 feet below low water, or 12 feet below the bed of concrete surrounding the tops of the piles, and on the south side to a depth of 11 feet 6 inches. Below this is a bed of stony brown clay ranging in thickness, according to the borings, from 2 feet at the north to 6 feet at the south side of the pier. Underlying the clay is a bed of freestone, of which the top layer for a depth of about 3 feet was shown by the borings to be somewhat broken up. The piles of the foundations are driven, according to the original plans, into the freestone, to a depth of about 20 feet below low water.

The original foundations were enclosed by coffer-dams formed of two rows of sheet piling 4 feet apart, with 12-inch by 12-inch king piles 4 feet apart and 12-inch by 6-inch sheeting between, with the sides grooved so as to fit into one another. The sand was excavated to a depth of 16 feet below low water, and the space was filled from that level to a few feet above high water with well-rammed clay. The king piles of the inner row were driven to the freestone, the intermediate piles, together with the whole of the piles of the outer row, being driven 5 to 6 feet into the clay. Some of the piles of the coffer-dams were of American rock elm. The coffer-dams were left in to protect the foundations, the inner row of piles being cut off 1 foot 6 inches above low water, and the outer row at low-water level. The space between the two rows

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was boarded over, the boarding being subsequently replaced by a bed of concrete.

In 1906 Messrs. Armstrong, Whitworth and Company approached the Railway Company and the River Tyne Commissioners with a view to widening of the outgoing or southernmost channel, in order to accommodate the very large battleships they were then building at their Elswick Yard above the bridge; and it was agreed to cut away a portion of the swing-bridge jetty which surrounds the centre pier of the high-level bridge and to remove the outer row of the coffer-dam piles on the channel side of the south pier, leaving the inner row of sheet piling to protect the foundations. The piles were cut off 9 or 10 feet below low water, as shown in Fig. 2, Plate 6.

Examination of Foundations.—In 1911 borings were made through the foundations of the three piers in order to test the condition of the material underlying the bed of concrete which, as previously mentioned, surrounds the tops of the piles. The borings indicated a certain amount of soft material along the north or channel side of the south pier, but not to an extent considered serious.

In 1915 it was decided to make a thorough examination of the piling of the three river piers, as far as this could be done with the assistance of diving-dress, and Mr. N. G. Gedye, M. Inst. C.E., previously Engineer to the River Tyne Commission, was commissioned to carry out this examination. Mr. Gedye's report showed that the sheet piling was in many cases out of line and out of perpendicular; that there were considerable gaps between the piles, amounting to 6 inches or more on the north face of the south pier; and that probings made on this face between the piles with an 8-foot probe indicated soft mud for its full length behind the piles. In the case of the north and centre piers it was not possible by probings through the coffer-dam to arrive at any definite conclusion as to the condition of the material underlying the concrete.

As a result of Mr. Gedye's report on the condition of the piling, the Author came to the conclusion that a thorough examination of the foundations of the south pier must be carried out; but after consultation with Messrs. Armstrong, Whitworth and Company, it was decided to postpone the work during the war, as it would necessarily involve a temporary narrowing of the channel, and in the case of a naval engagement it might have been necessary to bring large warships up to Elswick for repairs. It was therefore not till 1919 that it was decided to proceed with the construction of a steel pile coffer-dam round the south pier for the purpose of making a thorough examination of the foundations.

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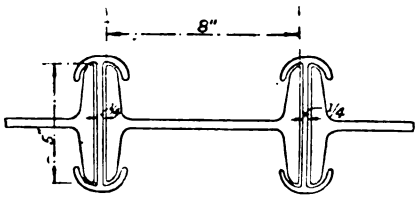
Underpinning of South River Pier.

Steel Coffer-dam.—The section of piling selected was "British Universal Joist" piling, the joists being 8 inches by 5 inches by 28 lbs., and the clutches weighing 15½ lbs. per linear foot (*Fig. 3*). On account of the narrow width available, a special pile-driving frame was constructed. This was 60 feet high, in order to deal with piles 52 feet in length and 12 feet 6 inches wide at the base; but on account of the narrow space between the line of the dam and the face of the pier the pile-frame leaders were fixed 7 feet 6 inches from the back of the frame (Fig. 4, Plate 6).

The dam was formed of 292 piles, including one special taper pile and one closing pile. It was found necessary to introduce a taper pile after driving about 80 piles, owing to the piles being out of plumb in a longitudinal direction in spite of careful wedging, but this was the only one required in the full length of the dam. The closing pile consisted of two tee pieces and two bulb flats. The

Fig. 3.



Scale: One-Eighth Full Size.

flats were riveted to either side of the web of one of the tees, the web of the other tee sliding between them. Two piles and two clutches were driven together by a No. 9 McKiernan-Terry double-acting steam-hammer weighing 7,700 lbs., the piston weighing 1,250 lbs. and striking up to 200 blows per minute. The steam-pressure required was about 90 lbs. per square inch. The driving with this hammer was very satisfactory, and no hitch of any kind occurred. The total time occupied in driving the dam, which was 208 feet 7 inches in length, was 100 working-days. The actual number of days driving was only 60, the balance being taken up in turning the pile-driver at seven corners. The average number of piles driven per day over the total time was three single piles, the average linear feet of single pile per day of 8½ hours being 59. The piles were driven on the north side of the pier to a depth of about 31 feet 6 inches below low water, and on the south side to about 25 or 26 feet below low water.

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The piles were driven between the two rows of piles of the original coffer-dam, where the latter existed, and immediately outside the inner row on the north side of the pier where the outer row had been removed.

Two 6-inch valves were fitted in the east end of the coffer-dam for emergency purposes. On the completion of the strutting these valves were closed, but the water inside the dam rose at the same rate as that outside, and it was eventually necessary to slacken the struts and to pass wire ropes with union screws round the dam in order to close the joints. This was immediately effective, and thereafter the water was dealt with by a 6-inch pump working intermittently.

While the dam was being formed along the east side of the pier there were indications that the piles were being driven through timber, and in carrying out the excavation at this side it was found that they had cut through an oak tree some 3 feet in diameter, which was lying diagonally under the foundation. One of the main bearing piles of the pier had been driven right through the middle of this oak tree; the pile was perfectly sound and did not appear to have sustained any damage whatever. The original sheet piles, however, where they had come in contact with the tree, were completely shattered.

Inspection of Foundation.—The dam having been made water-tight, the inspection of the foundation was begun. In the first place probings were made in the boreholes of 1911, and these indicated a greater depth of mud than had been present in that year. In considering these conditions and the method to be adopted in meeting them, it was thought possible that the removal of the semi-liquid mud might lead to a run of the waterbearing sand behind it and so cause danger by removing support from the piles and subjecting them to side pressure, especially in view of the heavy load on them. It was decided, therefore, to attempt to consolidate the mud by the injection of cement, and this was done through holes specially made just outside the concrete base. Borings afterwards made alongside these grouting-holes to test the effect of the grouting, showed that the grout had little, if any, effect in consolidating the mud, but was merely spreading out in a thin layer. When taken out in the course of excavation, the grout was found to be in a pasty condition. The result was disappointing, as it had been intended, if the grouting proved successful, to go into the matter of grouting the sand on a larger scale, so as to avoid if possible the more difficult course of underpinning.

The next step taken was to cut shafts through the base of the

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pier at six points, in order to obtain further information as to the condition of the mud. Three points were on the north, and three on the south side of the pier, the width of the shafts being 3 feet, the distance between the piles. On the north side examination showed that the amount and disposition of the mud was fairly uniform along the length of the pier. Against the steel piles of the coffer-dam the mud extended to a depth of about 16 feet below low water, or about 12 feet below the underside of the existing concrete. From this point it sloped up and died out under the pier at a point 15 feet from the steel piling. On the south side the inspection-shafts showed sand immediately under the concrete, with clay underlying it. In the south-west shaft clay puddle and brushwood were found under the concrete, suggesting that difficulty had been experienced with running sand in putting in the concrete.

The timber in the double 3-inch decking, the crown beams, and the piles, was found to be in excellent condition. The decking, where cut through by the shafts, was of oak and rock elm, and the crowns and piles were memel timber. There was no trace of creosote.

Underpinning of Pier.—The result of the examination from the shafts showed that underpinning was desirable, and possible if great care were exercised to avoid a run of the very unstable material, which would remove support from the piles. The total weight of the pier is about 7,000 tons, giving a load of approximately 58 tons per pile.

The work of underpinning was begun at the north-west corner, Section 1, Fig. 5, Plate 6. The distance apart of the piles was 4 feet from centre to centre, and the width of the heading was made 5 feet, in order that the shuttering might be fixed behind the piles. At a depth of 16 feet 6 inches below low water a strong foundation of clay was obtained against the steel sheeting and stepped up with the rise of the clay bed to the south. The depth of 16 feet 6 inches below low water is only 2 feet 6 inches above the points of the bearing piles. Section 2, Fig. 5, was next dealt with in a similar manner, and then the intervening Section 4, thus forming a solid block of concrete 12 feet wide across the north-west corner of the pier at what appeared to be the weakest point. With the experience gained at this corner, and the condition of the mud and the underlying strata being now known, the underpinning was carried out section by section without any hitch, the principle being that no two adjacent sections were dealt with at the same time, the concrete being allowed to set before the adjoining section was disturbed. With one exception the sections on the north side were completed before the opposite ones on the south side, but in

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one case they were done together, a heading being formed from one side of the pier to the other.

Fig. 5 shows the actual position of the piles immediately under the bed of concrete, 5 feet below low water. The piles, as they were exposed, were strutted in all directions every 5 feet in height. The bed of concrete surrounding the tops of the piles was in good hard condition, and it was not found necessary to remove it. The order in which the sections were undertaken is shown in Fig, 5, and a typical cross section of the underpinning is given in Fig. 6, Plate 6.

Fortunately the railway-company had a suitable piece of land available on the south side of the river, near the work, for use as a yard. A temporary bridge was erected connecting this with the dolphin in the river at the west end of the pier, and a staging was made to connect the dolphin with the steel coffer-dam (Fig. 7, Plate 6). Protective piling surrounded the coffer-dam, and on several occasions prevented ships from colliding with it. A 2-foot-gauge tramway connected the yard with the work at the pier, and excavated material was removed by this means, concreting material being conveyed to the work in the same way. Hand cranes were erected at both ends of the pier, but by suitable arrangement of pulleys these were worked by a pair of steam winches placed under the concrete-mixing platform at the west end, and supplied with steam from a boiler in the yard, since fires were not permitted on the timber staging. One man was able to attend to both winches.

The method of carrying out the excavation was as follows. Six shafts through the base of the pier, as previously referred to, together with one at each end, making eight in all, were used for removing the excavated material and supplying the concrete to replace it. Small hand buckets were employed to pass the material up the shaft to an excavating-skip with bottom door, the latter, when full, being hoisted by crane and tipped into a wagon on the tramway above. When sufficient material had been removed to permit of it, the excavating skip was lowered down the shaft, and the material was filled direct into it, thus eliminating one stage in the process. The tramway wagon, when full, was taken to the yard and tipped, the material afterwards being carted to a suitable place of deposit.

The king piles of the old timber coffer-dam were cut off near the bottom of the excavation, and the half-timber sheeting, which was shorter, was removed, except where it had a firm hold, in which case it was treated in the same way as the king piles. The clear distance between the king piles was 3 feet, and three 12-inch by 6-inch piles with a bird's-mouth joint were placed between, and

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connected by the same joint to the king piles. Although this joint kept the piles locked together for a few feet near the top, it was found on excavating that the piles were, in some cases, as much as 3 feet out of position. In almost every instance they sloped outwards, the toes of some of the piles being cut off by the steel sheeting.

The concrete used in underpinning was a 1 : 2 : 4 mixture, the material being screened river-gravel. A ½-cubic-yard batch mixer was fixed under the staging at the west end, feeding direct to a distributing-stage inside the coffer-dam. The mixing-platform being level with the top of the hopper, the labour in supplying material was considerably curtailed. From the distributing-platform the concrete was barrowed to the appropriate shaft, tipped down a timber shoot, and shovelled into place. Although a fairly wet mixture could be used near the bottom, it was necessary near the roof, where packing was required, to reduce the proportion of water in the concrete.

In carrying out the underpinning it was not possible to ensure an absolutely tight joint between the underpinning and the underside of the original concrete. Such a joint was considered important, so that in the case of the slightest settlement hereafter the weight of the pier would be transferred to the new concrete foundation. It was decided to force grout into the joint by means of compressed air. Channels were formed at intervals at the top of the underpinning by packing the concrete around a 2-inch pipe, which was withdrawn before the concrete had fully set. For grouting, a small mixing-tank containing cement and water was connected by a pipe with the above-mentioned channels, and the grout was forced into the channels by an air-pipe connected to the tank at a pressure of 30 to 50 lbs. per square inch. As a check on the travel of the grout from the line of main flow through the channel, the upper portion of Section 26 on the south side was left unconcreted while the portion of the under-pinning to the east of it was being grouted from a point close to the east end. At first the grout appeared on the face of the new concrete in Section 26 a few feet from the top, but as the cavities were filled, the grout rose to the joint between the new and the old concrete. Leakage of grout was prevented by puddle, and on removal of the latter after allowing time for the grout to set, it was found that a very good junction between the new and the old concrete had been obtained. The quantity of cement used for grouting the whole of the foundation was 21½ tons. Check measurements and bench-marks were tested frequently during the work, but no movement of any kind was observed.

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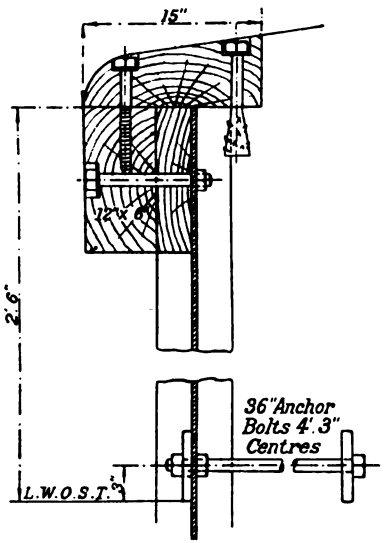
The portion of the steel coffer-dam above the base of the pier was cut off by means of oxy-acetylene cutting-plants. As the cut-off level varied from 1 foot 9 inches to 2 feet 6 inches above L.W.O.S.T., work was only possible during low water at spring tides, and then only for a short period. In the removal of the cut-off portions a pulley block was attached to the girders of the bridge, over which the winch rope was passed.

At each end of the north side of the foundation, 6-inch timber sheeting piles were driven to connect the steel coffer-dam with the original outer coffer-dam (Fig. 5). *Fig. 8* shows details of the timber

Fig. 8.



Scale: $\frac{3}{4}$ Inch = 1 Foot.

fender attached to the steel piles on the north side and the bolts securing the top of the steel piling to the concrete of the foundation. The space between the steel piling and the pier was finished off with a 3-to-1 concrete apron, the material used being $\frac{3}{8}$-inch crushed whinstone.

The work, which was at a level of about 3 feet 6 inches above low water, was carried out during short periods and partly at night. At first it was found difficult to obtain a surface which would withstand the strong wash of the tide within a short time after being deposited, but this difficulty was overcome by adding soda in the proportion of a handful to a bucketful of water.

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Examination of Centre and North River Piers.

On the completion of the work at the south pier it was decided to make an examination of the foundations of the centre and north river piers, as far as it was possible to do so without enclosing them in coffer-dams, in order to find out whether there was any sign of displacement of the material surrounding the tops of the piles immediately underneath the beds of concrete. For this purpose shafts about 3 feet square were cut through the bases of the piers down to the underside of the beds of concrete. In order that work might be carried out continuously without being affected by the tide, a special steel working-chamber was constructed, 4 feet by 3 feet, with rounded ends, and large enough for two men to work in (Figs. 9, Plate 6). This chamber was bolted down to the base of the pier, and from the top of it a circular shaft 3 feet 8 inches in diameter, fitted with a ladder, was carried up above high water.

Two examination-shafts were put down, one at the north-east and one at the south-west corner of the centre pier. These shafts went through the timber jetty of the swing bridge which surrounds the centre pier, and therefore no protective piling was necessary. In both shafts the timber and masonry cut through were found to be in excellent condition, and the sand underlying the concrete was found to be practically in contact with it, there being not the slightest sign of any removal by scouring.

Figs. 9 show that the conditions at the centre pier are more favourable than those at the north and south piers, inasmuch as the pier is protected by the swing-bridge jetty and the bed of the river on both sides comes well up to the top of the outer row of piles of the coffer-dam.

In the north pier three examination-shafts were sunk, at the north-east, south-east, and south-west corners, respectively, special timber staging being built to protect the steel shafts from damage by shipping. The examination showed the timber decking, crowns, and piles to be perfectly sound. It was not possible in this pier to control the flow of water in the shafts when once the concrete was broken through. It was therefore necessary to fit each shaft with an air-lock, the supply of air being obtained from the air-pipe of the electro-pneumatic signalling installation which runs over the bridge, in order to complete the cutting away of the concrete and to inspect the material underlying it. The pressure required was 8 lbs. per square inch at high water, and by this means a satisfactory examination was made possible. There was evidence of a very slight displacement of material immediately under the concrete on the

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south face of the pier—the channel side—the depth of black, sandy mud ranging from 3 feet at the west end to about 4 feet at the east end. This was not deemed sufficient to affect in any way the stability of the pier, but it was considered essential that the dam should be made thoroughly effective, in order to prevent any possible further washing out of the sand. This pier was therefore surrounded by a row of steel sheet piling of the same section as that used at the south pier, driven between the two rows of the coffer-dam piles, the piles on the channel, or south side being 30 feet 6 inches long, and on the north side 18 to 20 feet long (Figs. 9). The channel on the north side is quite shallow, and there is little risk of scouring. The lengths cut off the tops of the south pier coffer-dam piles were used for the shorter piles on the north side. The tops were finished off with a creosoted fender and capping, and a concrete apron, in the same way as at the south pier.

Costs.

The total cost of underpinning the south pier, including all temporary staging and temporary works, was £38,774 13*s.* 3*d.*, the cost of the main items of the work being as under :—

	£	*s.*	*d.*
Temporary staging	4,300	0	0
Steel coffer-dam—including driving, strutting, special piles, cutting off and removing upper portion	12,895	5	11
Excavation	4,170	0	0
„ timbering	1,255	0	0
Pumping, etc.	5,340	0	0
Concreting	5,360	7	4
Permanent fender and timber sheeting	450	0	0
Plant and stores, temporary offices, etc.	5.004	0	0
	£38,774	13	3

The unit prices were :—

Temporary staging	15*s.* per cubic foot.	
Steel coffer-dam	£10 15 0 per square yard.	
	£62 0 0 per linear foot.	
Excavation including timbering	£ 4 13 5 per cubic yard.	
Concreting	£ 4 12 5 „ „	

The work at the south pier foundations began in June, 1919, and ended in April, 1921, during which period labour costs reached their maximum, the rate for labourers being 2*s.* 1*d.* per hour between June, 1920, and March, 1921.

Messrs. Sir William Arrol and Company carried out the work on a time-and-material contract in a most satisfactory manner.

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Mr. J. T. Thompson, Assoc. M. Inst. C.E., acted as Resident Engineer for the Railway Company on the work, and Mr. J. H. Nicholson, M. Inst. C.E., as Agent for the Contractor; the Author wishes to acknowledge his indebtedness to them both for their valuable help in the preparation of this Paper.

The Paper is accompanied by seven sheets of tracings, from which Plate 6 and the Figures in the text have been prepared.

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Discussion.

The AUTHOR, after showing lantern-slides illustrating the work, referred to some samples of timber exhibited at the meeting, which had been taken out of the old foundations. Their excellent condition had justified the whole of the original timber being left in place. American rock elm was specially mentioned by Captain Laffan in his report upon the bridge to the Board of Trade in 1849, where the interesting fact was recorded that the rock elm piles had caught fire at the top when driven by a Nasmyth hammer. The Author did not know whether this timber was used at all now, but in an interesting book,[1] written by Thomas Laslett, an Admiralty Inspector of Timber, it was stated that 2,000 to 3,000 loads of this timber were imported into each of the London and Liverpool markets at that time; and the writer specially mentioned its durability under water. It was the slowest-growing timber known, only increasing in diameter 1 inch in 14 years. It would be interesting to hear if any other members had had experience of American rock elm. The Author.

Mr. G. E. W. CRUTTWELL remarked upon the forethought displayed by the original engineers and builders of the bridge. The river had originally been only 3 feet deep at low water, but the depth had increased since to 27 to 30 feet. That must be far deeper than anything ever contemplated when the bridge was built, and yet it was still standing and, after underpinning, was as good as new, if not better. He was surprised to see the wonderful state of preservation of the timber, especially the rock elm. His own firm had used that timber extensively up to the early part of the present century for fendering in front of jetties, etc., and they had found it very useful as a hardwood; it was tough and not liable to flake off, and it did not split. In recent years they had had to give up the use of it, on account of its cost and the difficulty of obtaining it. He could support the Author's opinion of its durability; and, if it could be obtained at a reasonable price, his firm would be only too glad to resume its use. It was stated (p. 176) that the steel sheeting around the south pier had been driven to a depth of Mr. Cruttwell.

[1] "Timber and Timber Trees." London, 1875.

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Mr. Cruttwell. 31 feet 6 inches. Fig. 2, Plate 6 showed that the surface of the yellow freestone was about 22 feet below low water. According to that, the piles were driven about 9½ feet beneath the surface of the yellow sandstone, on the north side of the south pier; it seemed curious that it should have been necessary to drive to so great a depth, and he imagined that the piles had been rather punished in the process. The Author had proceeded with great caution in taking out the underpinning holes. From Fig. 5 it appeared that quite small holes were first opened up, measuring about 35 to 40 square feet; but towards the end of the work the size was increased, holes Nos. 27 to 29 measuring 180 square feet. Possibly it had been found, as the work went on, that it was safe to increase the size of the holes. He found the average area of the thirty-one holes to be 87 square feet. The Author stated that no two adjacent holes had been dealt with at the same time, the concrete of one being allowed to set before the adjoining hole was disturbed. How long was it before the adjoining hole was taken out? In regard to the north pier, it was said that there was evidence of a very slight displacement of material immediately under the concrete of the south face of the pier, but that it had not been thought necessary to underpin or take any remedial measures, except to drive sheet piles around the pier. He thought it was rather a pity that sheet piles, which apparently came only up to low water, having been driven, steps had not been taken to make absolutely sure of the pier. He would like to have the Author's assurance that he considered the pier to be absolutely sound, for, on reading the Paper, it did not seem to him perfectly safe to leave it in that condition.

Mr. Savile. Mr. L. H. SAVILE remarked that the Author had been able, by his foresight, to anticipate what might have happened to the bridge and to take steps to find out whether it was happening. Having made his investigation, he had prepared a detailed scheme for dealing with the damage, which scheme had been carried out in a most satisfactory manner, with the result that the bridge had now been made safe. The work reflected the greatest credit on the Author and on those who had had the difficult task of carrying his proposals out in detail. It would have been interesting if the Author had added to Fig. 5, Plate 6, the actual positions of the shafts, as it would have been easier then to follow how the various sections had been dealt with in the order in which they were numbered. In view of the statement that in adjoining sections of the underpinning one had not been dealt with until the other had set, it was a little difficult to see how it was that the numbers of some of the adjoining

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sections were consecutive, namely, Nos. 5 and 6, Nos. 9 and 10, Nos. 21 and 22, and Nos. 31 and 30. A possible criticism of the method of underpinning concerned the making of the joint with the underside of the original concrete by forcing in cement grout. No doubt that probably made a satisfactory junction, but a better method might have been to stop concreting, say, 3 feet from the top, and, after that had set, to build the last portion in brickwork up to the original concrete. What had happened to the bridge showed the importance of allowing in any bridge of that sort a more than sufficient margin for scour, because, as Mr. Cruttwell had said, the engineers who built it could never have realized how greatly the scour would be increased by the blocking of the river by the piers of the bridge. In India, where he had had experience of bridges over sandy rivers, it had been often necessary to carry piers down 80 or 90 feet below the level of the sand, because in the rainy season probably 40 or 50 feet of sand would be alive. Mr. Savile

Mr. H. H. DALRYMPLE-HAY observed that the Paper was very welcome at the present time, because the subject dealt with had been in the public mind for close on 2 years in another connection. Therefore, as the matter was not only of local and municipal, but also of national and international interest, the Author was to be congratulated on having brought the Paper before The Institution. The method of underpinning described was one with which most engineers who had to do with that class of work were quite familiar. The way in which it had been carried out was most commendable. The Author discovered, as a result of his examination of the foundations, that on one side of the south pier there was soft liquid mud, then sand; these were overlying what was designated in Fig. 2, Plate 6, as "freestone ramble"—he did not know quite what that meant, and would be glad if the term were explained—and below that there was a bed of freestone. Captain Laffan's report was quoted as stating that there was 31 feet of mud and 4 feet of gravel, and then some clay, and below that hard stone. That report did not apply in connection with the south pier; and he assumed that it only applied to the centre and north piers. The strata sloped down towards the centre of the river, and for that reason the foundation of the south pier had not required such long piles as had the other river piers. Compared with the other two, the south pier was comparatively simple to underpin. He thought that in all the cases the designers had been fortunate in finding the hard "freestone ramble" on which to terminate the piles, because they had actually had to depend entirely upon the end bearing resistance of the piles Mr. Dalrymple Hay.

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Mr. Dalrymple-Hay.

upon a good bottom. Above that there was, comparatively speaking, soft material, and the whole of the top of the piles was bonded in good concrete. A bridge built on that system, therefore, stood as it were on a number of stilts, and if the work had been left simply in that condition, with no other protection, it might have been anticipated that trouble would ensue. The engineers therefore left the double coffer-dam in, to protect the foundation, which protection rendered the bridge perfectly safe; and he thought that, if that coffer-dam had not been removed, and even if the Author had not put in concrete, the bridge would have gone on carrying the load as it had done formerly, because as long as the sides were protected from scour, the load would remain wholly on the piles in that particular case, and no advantage would be obtained from the concrete. Those piles were very heavily loaded—about 58 tons per pile—and it was, no doubt, very wise to bond the whole foundation together and distribute the load over the ground, so that, if the piles failed, the load would be taken by the concrete itself. He was interested to see that the piles were in good condition. His view was that, if timber was below water, it would last practically for all time: that had been found in the present instance, and the fact ought to be noted by every engineer. He knew of other similar cases, and members who had larger experience than he had found the same thing to be true. In underpinning foundations of the type described, consideration should be given to providing lateral support to the pier rather than taking the load at the bottom. By lateral support he meant protection against the possibility of scour. The steel dam had been driven well below the level of the foundations. There was no doubt that the method of excavation adopted had been economical and right in the particular case of the south pier; but, if the Author attempted to make an open excavation at the north or centre piers in 36 feet of mud, he would find it impracticable: the result would be a high head of water outside the dam and nothing inside to counteract it, and the dam would have to be carefully strutted and would be unduly costly. The obvious way to overcome that difficulty was to apply the elastic reaction of compressed air in a closed chamber to balance the water-pressure outside against the dam, and render the foundation capable of being excavated. With the open type of excavation, while excavation was proceeding, the clay was loosening itself, and when the load came on there was a small settlement, which all engineers tried to avoid. That was due to the fact that the clay had been breathing up from exposure. With a foundation for which it was not necessary to make a wide excavation, it was better to use compressed air,

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so as to limit the work to as small an area as possible, because more load could be put on a clay foundation which had been excavated under compressed air than on one which had been excavated in the open, with rain killing the clay at the top. With regard to the scour, he did not quite follow from the Paper whether the depth of 27 or 30 feet below low water was attributable to scour proper, or whether it had been dredged at the request of Messrs. Armstrong, Whitworth and Company. Stephenson knew perfectly well that he was obstructing a portion of the river and would get scour, and he left the coffer-dams in. The coffer-dams had since been taken out, but Mr. Dalrymple-Hay was glad to see that the steel piles remained, because that was the saving point of the whole scheme. Mr. Dalrymple Hay.

Dr. OSCAR FABER observed that the steel sheet piles seemed to have been extremely successful. He had had occasion to use various types of steel pile, and he had found that the type mentioned by the Author was particularly successful, whereas several other types had proved unsatisfactory. The method of securing water-tightness with wire ropes and tightening-screws was interesting. It seemed to him that the forces which could be brought to play on so large a dam constructed in that manner would have been relatively so small, that he could not have hoped to make it water-tight in that way; but apparently it had been successful, and that experience was useful to have on record. In driving such piles it had been possible sometimes to omit the wedge closing-pile, which was referred to in the Paper. That required extraordinary skill and care on the part of the contractor, and he thought the arrangement described in the Paper was probably better and safer, though on smaller foundations such closing piles had occasionally been omitted successfully. In that case the piles were driven in groups, of perhaps five at a time, in rotation, each group being driven only a little way down at one time. With regard to the order of concreting the various sections described in Fig. 5, Plate 6, why had all the sections on one side been concreted before those on the other? He would have been inclined to fear that, in case of any settlement or running taking place, it might be rather dangerous to have one side of the pier supported while the other side was entirely unsupported. Why had not a few sections been completed on one side, and then a few on the other, so as to keep the pier balanced? Had any temperature-measurements of the large masses of concrete been taken? It was his experience that with modern cement, which contained much more chemically active material than the old cements, there was, when dealing with large masses of concrete, a very considerable rise of temperature when the concrete set under Dr. Faber.

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Dr. Faber. circumstances where the heat could not easily get away. In an underpinning job there was very little surface for radiation or for dissipation of the heat by any other method. At the Bank of England the outer walls were being underpinned with concrete 8 feet thick and 50 feet deep, and in the previous week he had had temperature-measurements taken systematically of the concrete when it was placed, and subsequently at 3-hour intervals; 3 days after placing the concrete the temperature had risen to 100° F., from 50° F. at the time of placing. The cement was not a quick-hardening cement, with which undoubtedly a still greater rise of temperature would occur. He thought the low cost of the underpinning work was a matter on which the Author deserved congratulation; and it was particularly opportune because, as was generally known, proposals for the underpinning of other bridges were under discussion, in which very much larger figures had been mentioned.

Mr. Gedye. Mr. N. G. GEDYE mentioned that in the library of The Institution there was a set of drawings—made, he believed, 2 or 3 years after the high-level bridge had been completed—which were some of the finest drawings of an engineering structure he had ever seen. They appeared to show the foundations of the bridge and the surrounding levels substantially as they were on completion, and from that point of view they were very valuable. He had been closely connected with the early stages of the matter covered by the Paper, first as the Engineer to the Tyne Improvement Commission and later, after he had left the service of the Commissioners, when he was instructed by the North Eastern Railway Company to make a detailed examination of the foundations. In the summer of 1910 he had occasion to examine certain periodical soundings of the river in the neighbourhood of the bridge, and was rather concerned at the depth of water near the coffer-dam of the south pier, where it had been cut away 4 years previously. About that time also, small damage had been done by some craft to the timber work of the south pier, and the Tyne Commissioners had been asked to send a diver to examine the damage for some party concerned. The Tyne Commission diver who carried out the inspection told him something of what he had seen, and, as the result of that and of his knowledge of the soundings, he went down himself in diving-dress and found the state of affairs so serious that he reported it to the Chairman of the River Committee, and on his authority he went to the late Mr. Charles A. Harrison, M. Inst. C,E., then Chief Engineer of the North Eastern Railway, and gave him all the particulars in his possession. As a result, Mr. Harrison caused the borings through the foundations to be made in 1911. In 1910 the conditions were very similar to those

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shown in Fig. 2, Plate 6, but that section did not show what he then discovered, and confirmed in 1915, namely, that in more than one place on the north side of the south pier foundation the mud—called by the Author "black sandy mud," but which Mr. Gedge preferred to call black slurry and water—was so liquid in places that it could be swished about with the arm; and in one place he could get his leg through the gaps in the piles. In 1910 an absolute void extended under the concrete to 6 feet below low water for some distance along the north face of the pier, and in 1915 he found that that void had extended down to the point where the words "bed of river" were shown on Fig. 2; that was, the void had increased from 6 feet to 10 feet in depth. In 1910 he used a 4- or 5-foot probe, and in 1915 an 8-foot thin steel rod, and he was able to probe in twenty or twenty-five places along the side of the old coffer-dam. He also touched, with the point of the probe, many of the piles in the second row under the foundations, and even some in the third row; in several places he could feel the underside of the concrete with the probe. As far as he could reach with the probe he found the material was so soft that by pressure of the wrist he could move the rod about. He had no doubt that that black liquid and sandy mud had replaced the sand which was mentioned in 1849 by Captain Laffan, who described the concrete as being founded on loose sand which extended to a considerable depth. That sand had no doubt been there, and probably was under the north and centre piers to this day. On the north side of the south pier the gaps in the inner skin of the coffer-dam piling, being protected during 60 years by the outer skin and the puddle clay filling, had done no harm at all, but as soon as the outer skin was removed in 1906 the open gaps let the sand out. It was a case of infiltration of liquid mud, replacing sand. His figures of the levels of the underside of the concrete, as obtained by probing, agreed very closely with the levels in Fig. 2; in two or three instances he had noted the underside of the concrete as being at a slightly higher level. A point which had puzzled certain speakers was the level of the river-bed. In 1849, when the bridge was built, the range of tide at the high-level bridge was very much less than the present range of 15 feet. At that time, on account of the obstruction of the river lower down by the old Tyne bridge, which had very short spans and was a structure resembling in some ways old London Bridge, the level of high water was about 3 feet higher than at present, and low water was 5 to 6 feet higher than the present low water. The bed of the river in the centre of the channel, as shown on contemporary drawings, was practically at the level of the top of the coffer-dam shown on the Author's

Mr. Gedge

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Mr. Gedye. diagram. In no part of the channels between the piers was there more than 3 or 4 feet of water at the then low water. The deepening of the channels was not due to dredging in the neighbourhood of the bridge. During the time of his predecessors and during his own time and since, no dredger had been ever allowed to go near the high-level or swing bridges for dredging. The state of affairs was due entirely to scour. The old Tyne bridge was removed about 1870 and replaced by the present swing bridge, which was completed about 1877. The lowering of the water-levels had been brought about by the removal of the obstruction caused by the old Tyne bridge and the deepening of the river-bed by dredging below the swing bridge and above the high-level bridge. The river above Newcastle was not dredged until after the swing bridge was opened. The Author mentioned 27 to 30 feet as the present depth of the channels at low water, but in 1915 there was actually in the north channel one sounding of 36½ feet at L.W.O.S.T., where in 1849 the bed was about 2 feet above the present low-water level. He had been able to satisfy himself in 1915, when he made the underwater examination, that there was no reason to doubt the soundness of the piles. In addition to the piles which could be touched by the hand on the inner skin of the coffer-dam and in the outer rows of coffer-dam piles, he was able to probe twenty or thirty piles inside, and in no instance could he feel any sign of decay: he did not think he could prick the point of the spear into the timber more than ½ inch. He was not surprised, therefore, to see the beautiful samples of wood which the Author had shown that evening. The section of elm was from one of the sheeting piles which were bird-mouthed into king piles, but 5 or 6 feet from the top of the coffer-dam a very large proportion of bird's-mouths were out of joint. At the level of the river-bed very few piles were perfectly jointed; the probe would go through half of them. The north and centre piers had been protected to some extent from scour by the banks of slag which had been deposited alongside the actual coffer-dams. In the case of the centre pier that had been possible without risk of encroaching on the channel, on account of the extra width given by the presence of the swing-bridge jetty. The presence of that slag accounted for the comparatively steep slopes of the river-bed at about low water. In 1915 he calculated that the maximum pile loading was not far short of 64 tons, making allowance for the increase of the live load and additional dead load since 1849, and for the effect of wind-pressure. Captain Laffan's report of 1849 gave particulars of loads which indicated an average load of 51¾ tons per pile. His own estimate of 64 tons probably agreed very closely with the Author's

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figure of 58 tons per pile, which was given as an average value. It was extraordinary that in spite of the bad state of one side of the pier there was no settlement, and the Author gave no indication of any settlement having taken place. In 1915 there appeared to be 2 inches difference of level between the bench-marks of two iron datum plates, one in the north pier and the other in the south pier. The former was found to have its mark exactly at the level of the old Tyne datum, which had been in use on the Tyne from at least as far back as the sixties. It marked a certain level 7·92 feet above Ordnance datum, one that was known to everybody and no doubt would have been set accurately. When the plate had been put in he did not know. The one on the south pier was just 2 inches lower. Presumably those plates, which were of the same pattern, had been put in on the same date. Had the Author been able to find out the reason for that difference, and whether there had been any settlement in the south pier relatively to the north pier? The statement of costs (p. 183) was of interest and value. Did the totals given include all charges for supervision and contractors' profit? Having regard to the high costs which obtained at the time when the work was carried out, the total cost must be regarded as very moderate. He congratulated the Author on a valuable contribution to the Institution's Proceedings, possessing both historical and engineering interest. Mr. Gedye.

The PRESIDENT remarked that he thought it fitting to allude to the great high-level bridge which was a triumph of their engineering predecessors, the Romans, the Pont du Gard, near Nimes, Provence. That magnificent structure, consisting of three bridges one above another, had stood for practically 2,000 years, and to-day was a finer instance of Roman architecture than anything that could be seen in Rome. The President

The AUTHOR, in reply, stated that the length of the steel piles as ordered was 52 feet, and this provided for the piles standing 5 feet above high water and being driven below the bed of the river. The piles on the north side of the pier had been actually used of the full length, but on the south side it had not been possible to drive them down so far. The driving at the finish had been very hard, sometimes requiring 200 blows to the inch. He attributed the watertightness of the dam very largely to the fact that the piles had been driven down so far. They had not been damaged by the hard driving, and the lengths cut off had been used for the protection of the north pier. With regard to the sections, as the work proceeded it had been considered quite safe to increase their size, and in some cases, owing to the irregularity of the piles, the sections would have The Author.

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The Author. been rather difficult to deal with if they had been limited to the space between two rows of piles. The concrete had generally been allowed to set for 6 or 7 days before an adjacent section was opened up. The cavities under the north pier were very small, and he believed one of them had been formed by the material being blown out when the compressed air was first applied. He was satisfied that they were negligible and that it was unnecessary to do anything to them. The shafts had been sunk three on the north and three on the south side of the south pier, about 20 feet apart, between the inner row of the coffer-dam piles and the first row of the bearing-piles of the pier, and headings from these shafts had been driven between the first and the second row of bearing-piles to give access to the other sections. An additional shaft had been put down later at each end of the pier. With regard to the numbering of the sections, although Fig. 5, Plate 6, showed generally the order in which the sections had been undertaken, the whole of the excavation or concreting of a section had not in all cases been carried out continuously, and work had been going on at the same time in other sections, but in all cases sufficient time had been allowed for the concrete to set before the excavation in an adjoining section was taken in hand. He was quite satisfied that the grouting under compressed air had thoroughly filled the joint between the old and the new concrete; the special test at the east end (p. 180) had proved that the grout was travelling well and filling up all interstices, including the joint between the old and the new concrete. All the piers were protected against scour by sheet piling which extended approximately to the bottom of the river-bed, and in addition slag had been tipped on the sides of the channels adjoining the piling. Soundings taken over a period of years showed no deepening of the river below the depth to which it had been dredged on each side of the bridge. No dredging had actually been done at the bridge, but the channel had been dredged to 25 feet at low water up to a short distance on each side of it, so that the present depth was directly due to dredging. Referring to Mr. Dalrymple-Hay's remarks, Captain Laffan's description of the river-bed undoubtedly referred to the formation at the centre of the bridge. The word "ramble" or "rammel" was used in the North for a loose friable rock, and was often applied to the thin bed of shale lying above a coal-seam. The word was used in a great many counties, but generally in the form "rammel." Borings in the north of England were generally carried out by men who did boring at the collieries; hence the use of this word. He agreed that the underpinning of the south pier was a much easier work than the north or centre piers would have been; but if he

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had had to deal with the latter he would have considered seriously the question of carrying out the work without compressed air, especially in view of his experience of the south pier dam. In reply to Dr. Faber, the concreting of the sections on the north side had been undertaken first because the condition of the material was so much worse on that side—the sand under the south half of the pier was comparatively dry, and there was little difficulty in controlling it. No temperature-measurements of the concrete had been taken. Referring to Mr. Gedye's remarks as to the deepening of the channels, although no dredging had been done actually between the piers of the high-level bridge, the channel was dredged through the swing bridge to within 50 yards of the high-level bridge on the east side in 1912, and as early as 1877 dredging was carried out on the west side to within 80 yards of the bridge. The old Tyne bridge was removed in 1874–75, and the swing bridge was completed in 1876. A good deal of dredging was, however, done above or west of the bridge before that date, commencing in 1871 and continuing ever since; therefore, although the short length of 130 yards through the bridge had not actually been dredged, the deepening of the channels was undoubtedly due to dredging. As to Mr. Gedye's surprise at there being no settlement, it must be remembered that the piles were dependent for their bearing power on the fact that they were driven down to a rock bottom, and not on skin-friction. They were tied together at the top by a bed of concrete, and their stability would be affected but slightly, if at all, by a slight change in the consistency of the material surrounding a short length below the concrete. There had not been the slightest sign of settlement. How long the plates referred to by Mr. Gedye had been out of level he was unable to say, but there had certainly been no settlement during the 35 years he had known the bridge. Any settlement would have been noticed at once on the permanent way on the bridge. As to the cost of the work, the totals included all charges for supervision by the contractors and the contractors' profit, but did not include anything for supervision by the Company's engineer and the resident engineer.

The Author.

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Correspondence.

Mr. Pratley. Mr. P. L. PRATLEY remarked that the underpinning of the south pier constituted a very interesting piece of exploratory and repair work to an old structure. It struck him as important to know—as the Author and those acquainted with the Tyne doubtless knew—the actual cause of the deepening of the river from 3 feet to 30 feet, and the length of time that change had been in progress. Had it been more or less continuous and uniform since 1846, and due principally to natural scour, or had it resulted from definite operations at known dates? Arising from the answer to this point were two further questions: what had given rise to anxiety or suspicion regarding the foundation, and whence had come the mud which had apparently replaced the sand originally left under the concrete footing? From the Paper it appeared, of course, that long before 1906 some navigation requirements had necessitated a deepened channel between the centre and the south piers, where the clear width was never more than 100 feet. Seemingly that channel was at first kept quite narrow, out of consideration for the pier foundation, but in 1906 the shipping needs became more insistent, and a distinct inroad was made on the protection works originally built around the footing. No subsidence or cracking of masonry was alluded to, and he wondered what had prompted the 1911 investigations. Incidentally, it was to be presumed that the bore-holes then made were subsequently plugged. There would seem to be no reasonable doubt but that the normal disturbance of the water, due to current and tide, aggravated by the passage of vessels occupying a considerable portion of the actual cross section of the channel, had resulted in a washing movement under the footing concrete, and a carrying away of the sand and clay, with simultaneous or subsequent deposition of mud similar, most likely, to that covering the river-bottom. Such a strong wash was spoken of by the Author as having caused trouble during the pouring of concrete. It was fairly evident that no serious portion of the load on the pier had ever been carried by skin-friction of the piles or by bearing of the concrete, and that the piles acted as just so many struts carrying the load to the rock, well tied at the top and steadied at the foot by the broken and dislodged fragments of stone, which he gathered to be the meaning of the term "ramble" (Fig 2, Plate 6). He felt that in similar circumstances to-day, concrete down to rock would be the natural method adopted for a pier carrying more than

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7,000 tons. The load per pile, even on 12-inch by 12-inch squared timber, was much higher than that ordinarily allowed in his practice; and the fact that 75 years of efficient service had not impaired the usefulness of the piles spoke well for the value of the timber used. Memel timber was a term unfamiliar in Canadian practice, but it was presumed to be a species of Baltic fir. Information on that point would be appreciated. It would also be of interest to know whether the piles had been shod with iron for driving into the freestone. The fact that a water-logged oak had been driven through without the pile suffering damage seemed to indicate that the piles had been shod, as the sheet-piling, which would be unshod, was so completely shattered. The Author's note as to the absence of creosote suggested the inquiry whether it had been considered at all probable that signs of such impregnation might be found, and whether it was known that such preservative was recognized as early as 1846? The type of steel sheet piling used in the work was new to Mr. Pratley, and at first sight seemed to offer no advantage over the lighter Lackawanna type. With clutches it weighed about 62 lbs. per square foot of dam, against 35 lbs. for the 14-inch arch type, which was in extensive use in Canada, several hundred tons having been put into Montreal harbour bridge up to date. While each piece had been lighter to handle, this advantage would seem to have been more than balanced by the fact of having to handle more pieces per linear foot of dam, and having to select two kinds alternately. For these or other reasons the piling had not driven tightly, and assuredly had driven very slowly. Penetration should have been easy, and the equipment as described seemed first-class, yet three piles per day, or about 40 square feet, seemed ridiculously slow. With substantially the same hammer, namely, a McKiernan-Terry of 6,850 lbs. operating at 70 blows per minute, the whole of the sheet piling on four sides of a 24-foot by 82-foot pier coffer-dam had been driven on an average 25 feet into hard gravel with occasional boulders in 8 working-days. On the neighbouring pier, with a little easier penetration, slightly more linear feet of the U.S. Steel Corporation type of piling had been driven in 9 working-days. He felt that some cause, not mentioned in the Paper, must have been operating to slacken the rate of driving of the steel piling. The actual underpinning seemed to have been carried out very carefully and thoroughly, and the resulting foundation should prove practically permanent. The protection afforded by the steel sheet piling was such as completely to confine the clay bed under the concrete, so that virtually an immovable base down to rock had now been provided. The cost did not seem unduly high, if the cost of

Mr. Pratley.

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Mr. Pratley. materials in 1919–21 had been on the same high plane as in Canada. The sheet piling worked out at about 5*d.* per lb. in place, and the concrete at about £4 10*s.* Those figures could be considered reasonable under the circumstances. The wet excavation at £4 13*s.* 5*d.* per cubic yard was the only item that seemed excessive, and probably the difficulties of access, etc., which had to be seen to be appreciated, were greater than they appeared from the Figures. The expression "time and material" contract, he took to be equivalent to "cost-plus," with either a percentage or a fixed amount for overhead charges and profit.

Mr. Saner. Mr. J. A. SANER considered that the account of the work in connection with the south pier was most instructive, and all concerned were to be congratulated on the painstaking investigation and successful completion. The only criticism he had to make was that it would have been well to complete the underpinning of the north pier at some convenient time, before the new steel piling had been materially weakened by corrosion, which, as it was exposed to sea-water, would be fairly rapid. It might be of interest to record that when, about 50 years ago, the navigation of the river Weaver was being improved, it was found necessary to underpin two of the piers carrying the Dutton viaduct of the original Liverpool—Birmingham Railway (now the London Midland and Scottish Railway) across the river. That viaduct was designed by George Stephenson and completed by Robert Stephenson. It was built of Cheshire red sandstone, and had twenty-two arches, each of 60 feet clear span and 64 feet headway from water-level to the underside of the keystone. The foundations were laid in the Keuper marl formation. Without any interruption to the railway-traffic two of the piers had been underpinned with blue brickwork, and an inverted arch had been constructed so as to provide for a navigable depth of 15 feet. That work still remained perfect, notwithstanding the great increase in the weight and quantity of traffic using the viaduct.

Mr. Thorpe. Mr. W. H. THORPE remarked that he had found American rock elm, which had been used in the construction of a landing-stage to a sea pier in the Isle of Wight, sound and in excellent condition. Iron gratings resting upon the timber had, owing to movement induced by waves, worn into the timber to an extent which indicated many years of service, but without splintering or disturbance of fibre. The timber was situated under the most trying conditions as to alternate exposure and submersion.

Mr. Walker. Mr. E. G. WALKER observed that the historical associations and the importance of the high-level bridge as a link in the east-coast

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railway-system lent added interest to the execution of the works described in the Paper. It would have been instructive if the Author had plotted upon one of the cross sections of the river-bed the water-levels of the river as they existed at the time of the construction of the bridge; but it was evident from the figures given in the Paper that the development of the waterway of the Tyne had been vastly in excess of what had been contemplated at that time. The problem which had had to be solved in the present instance differed materially from that in one or two other instances in which the stability of the piers of important bridges had been under consideration. The most important distinguishing feature of the present case was that no incipient failure had taken place before the new works were put in hand, and the problem had been somewhat simplified by that fact. Secondly, it would appear from the Paper that the timber sub-structure remained in good condition. The load of 58 tons per pile would seem quite high unless the basis of calculation were kept in mind, namely, that the whole 7,000-ton load was carried on the piles, without any assistance from the material inside the original coffer-dam. Naturally, it was impossible to say how much support was given from time to time by the soil; it must have been a decreasing total as time went on. But it was of assistance, in visualizing the conditions, to look at the case from the opposite abstract point of view. Taking the total area of the concrete base of the pier, so far as could be ascertained from the Figures accompanying the Paper, the intensity of load over the area of the concrete was only about 2·65 tons per square foot. It would appear from Fig. 2, Plate 6, that in the original state, with a fairly tight coffer-dam and a shallow river-bed which would give little inducement for the sand inside the dam to flow out, the load was light enough to have made it almost possible to support the pier without any piling. Hence, the original foundations must be considered as amply strong for their purpose, and it was improbable that, in the earlier years of the life of the bridge, so large a load per pile as 58 tons existed. Fig. 2 gave the impression that, but for the cutting of the outer line of sheeting of the original coffer-dam in 1906, there would have been no necessity to underpin the south pier, at any rate for a long time to come. At the same time, it was quite possible that if the more recent conditions had remained for many years, a dangerous situation would have arisen, and the works would have been very much more difficult to carry out. His impression was that, although operations had had to be carried out in a very confined space, and, therefore, had been correspondingly difficult, the underpinning could have been carried out without risk at any

Mr. Walker.

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Mr. Walker. stage of the work, because it was possible to build a tight coffer-dam of steel sheeting down to bedrock and so prevent the outflow of any of the softer materials overlying the freestone. There would thus have been ample continuous support for the pier during the whole time that the various sections were being excavated. The measures which had been taken should be ample to ensure the safety of the structure for so long as the piers remained. The Author's attempts to grout the wet mud and sand underlying the pier were of interest and showed definitely the impossibility of forming a hard mass by grouting under such conditions. It was of value to have that experience on record.

The Author. The AUTHOR, in reply, remarked that he had given particulars of the dredging of the river in the vicinity of the bridge in the reply to the discussion (p 195). The borings in 1911 were made as the result of information given by the engineer of the River Tyne Commissioners with regard to the deepening of the south channel. The timber described as "memel" was *Pinus sylvestris*, or Baltic redwood. All the piles, both bearing and sheeting, had been shod, the former with heavy pointed shoes and the latter with chisel-pointed sheet-pile shoes; the 12-inch by 6-inch sheet piles had failed where they met the oak, owing to their inability as columns to withstand the driving. He had mentioned creosote because the date of the construction of the bridge coincided approximately with its introduction—at any rate, in the north of England. The figure of three piles per day, given as the rate of the pile-driving, was the average number over the whole period of 100 days; the actual driving time was 60 days. The driving in the solid freestone, especially on the south side of the pier, had been exceedingly hard; as soon as the piles had taken their bearing, the dam had been practically watertight. The expression "time and material" contract meant actual cost with a fixed percentage for overhead charges and profit.

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26 January, 1926.

SIR WILLIAM HENRY ELLIS, G.B.E., D.Eng., President, in the Chair.

The PRESIDENT said he had the pleasure to announce that H.R.H. The Prince of Wales had accepted the invitation of the Council to dine with the members on Wednesday, the 10th March.

The following Paper was presented for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

(*Paper No. 4562.*)

"Rugby Radio Station."

By ARTHUR STANLEY ANGWIN, D.S.O., M.C., B.Sc., and THOMAS WALMSLEY, B.Sc., Assoc. MM. Inst. C.E.

THE construction of the Government wireless station at Rugby was decided on to provide a means of radio telegraph communication of world-wide range throughout the whole of the 24 hours, thus avoiding the necessity of interposing intermediate stations in order to communicate with Australia and other parts of the Empire, and giving a sufficient margin of power to be effective both by day and by night and during conditions which are unfavourable for radio reception. To fulfil these conditions, the design of the station provides for high-frequency valve generators capable of an output of 500 amperes into an aerial supported on masts 820 feet in height.

The station was designed by the Wireless Telegraphy Commission and constructed by the Engineering Department of the Post Office.

Choice of Site.—The selection of the site was governed by the following considerations :—

(1) Absence of large objects likely to produce screening effects. A fairly level site is thus desirable.
(2) Sufficiency of water for cooling purposes.
(3) Suitability of the sub-strata to withstand the heavy concentrated loads of the masts.
(4) The presence of good roads and a railway-line for transport of material.
(5) Proximity to a main telegraph route, to minimize the cost of control-circuits.

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(6) The possibility of obtaining a bulk supply of electrical power at a reasonable cost.
(7) Reasonable nearness of a medium-size, town to provide homes and social amenities for the staff.
(8) Strategical considerations in the event of war.

The site finally chosen, as meeting these various requirements, is situated in the parish of Hillmorton, near Rugby, Warwickshire. About 900 acres, sufficient to accommodate a sixteen-mast aerial-system, were purchased, on a site having an average altitude of approximately 340 feet above sea-level, and a maximum variation, from the average level, of about 30 feet. To provide cooling-water for valves a reinforced-concrete reservoir, holding 500,000 gallons, has been constructed. This receives water from a stream that crosses the site and joins the Avon at Rugby. The third requirement is satisfied by the fact that the sub-stratum a few feet below the top alluvium is blue clay of more than ample thickness. Transportation of material is facilitated by the presence of good roads skirting the east and south boundaries of the site, whilst the London Midland and Scottish Railway is within easy reach. Any additional transportation facilities that might be required are provided by the Oxfordshire canal, which fringes the west of the site. For the purpose of control of the transmission-plant, use is made of the main telegraph trunk lines. Power is supplied by the Leicestershire and Warwickshire Power Company.

General Description.

Layout.—The general layout of the station is shown in Fig. 1, Plate 7. The main power and transmission buildings are placed almost at the centre of the site. To the south of the main buildings, eight masts, spaced 440 yards apart, form the corners of an irregular octagon, whilst the remaining four masts are erected to the north of the buildings. These four masts have been so spaced as to permit of the erection, in symmetrical arrangement, of four more, should such a course be decided upon.

Buildings.—The main power-plant is housed in a steel-framed brick-filled building, 185 feet 3 inches by 47 feet 3 inches by 32 feet 6 inches in height, with a flat roof over which the earth return wires are carried to form a screen. The valve transmitting-plant and the inductances are accommodated in a brick building 103 feet 1½ inch by 42 feet 4½ inches, by 59 feet high, parallel to the power-house. The inductance-room is over the valve-room, the latter being 20 feet 9 inches high. The quantity of steelwork utilized in this building has been restricted, in order to minimize losses due to the

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proximity of the high-frequency coils. For this reason the inductances are supported on solid wooden beams of Douglas fir, 22 inches by 9 inches, spanning the width of the inductance-room. The administrative offices are situated in front of the valve-room.

Masts.—The twelve masts are identical in form, each consisting of three vertical lines of posts, pitched 10 feet apart horizontally. In horizontal section, the posts thus form the corners of an equilateral triangle (Figs. 2, Plate 7). The system of K bracing adopted is more symmetrical than single diagonal bracing, and is better adapted to resist twisting of the mast. The bracing at each stay-point is strengthened. The ideal section of post is that which offers least effective area to wind, and, per unit of material, the greatest mechanical resistance to combined compressive stress due to loading and bending. The circular section is thus the ideal. Considerations of cost have hitherto operated against the use of a circular section for braced mast structures. The section employed at Rugby, which consists of two channels fastened together by means of a bent bar plate, is readily made up from standard channels, and has already been used at other wireless stations. The ratio of its greatest to least radius of gyration can be made near to unity. In the case of the Rugby masts this ratio, at the largest section of post, is 1·2 : 1. The posts are made up in lengths of 27 feet, all joints being connected by turned steel bolts fitting accurately in drilled holes. Joints made on the ground on the site, or in the works, are riveted.

The base of each mast terminates in a tripod, the lower extremity of which forms a socket seated on the spherical surface of a cast-steel plate (Figs. 3 and 4, Plate 7). Each mast is thus capable of free movement about its lower extremity. The great advantage of the pivoted mast is that the stresses in mast section and stays are more determinate than in the non-pivoted design.

The construction of the top of the masts is shown in Figs. 2 and 5, Plate 7. To support the aerial, a horizontal arm, carrying a sheave at each end, over which the aerial-halyard runs, projects beyond the mast. The arm is supported on rollers and, together with the handrail which moves with the arm, is free to rotate in a horizontal plane. The arm is thus able to take up the direction of the horizontal pull due to the aerial. Under the main supporting arm additional attachments are provided for the purpose of making, if required, fixed connection of the aerial to the mast. A lift to hold three persons forms part of the equipment of each mast. Each lift is provided with a mechanical safety-device and an electrical limit switch. As an alternative means of ascent, a ladder is fixed to the inner side of one of the posts. To raise the aerial and lift, a winch with two drums is fixed on the steel gallery near the base of each

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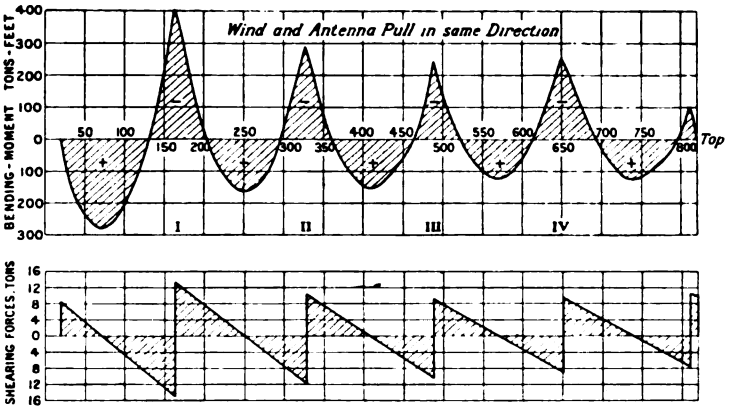
mast (Figs. 4). A 440-volt, three-phase, 20-HP. motor drives each winch. A feature of interest is the slipping device. The aerial-drum operates through a spring-loaded friction attachment (Figs. 6, Plate 8). The springs are so adjusted that, when the pull on the aerial-halyard exceeds 10 tons, the drum slips through the "Ferodo"-lined grips, thus letting out the halyard and relieving the tension.

Stays.—Fifteen stay-ropes, arranged in five groups of three stays, are provided for each mast. The angle between the horizontal projections of adjacent stays at each point of attachment to the mast is 120 degrees. Details of the stay attachment to the mast are shown in Figs. 7, Plate 8, whilst particulars of the lower end fixing to the anchor-blocks are shown in Figs. 8, Plate 8. Each concrete anchor-block has a mass equal to $1\frac{1}{2}$ times the vertical component of the calculated maximum stay-pull at the anchor.

Fig. 9.



Each stay has a steel frame near the anchor-block, which, in addition to holding porcelain insulators to insulate the stay from earth, also houses a small hydraulic pump. A pressure-gauge is connected to the pump, and, when correctly calibrated, reads the tension in the stay at its lower end.

All the mast-stays, which were made on the site, consist of straight, No. 10 S.W.G. galvanized high-tensile steel wires having a breaking stress of 100 tons per square inch.

Mast- and Stay-Stresses.—The maximum compressive stress allowed in the mast-posts is such as to comply with the formula $18{,}000 - 80\ l/r$ and not to exceed 14,000 lbs. per square inch, where l is the length of post between horizontal bracings, or distance between stays, and r is the least radius of gyration of post, or mast, section. The masts were designed to withstand a horizontal aerial-

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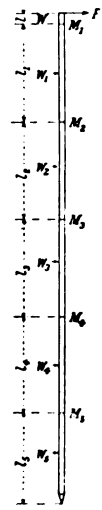
pull of 10 tons and a horizontal wind load of 60 lbs. per square foot over the whole of the exposed surface of the mast, taken at right angles to the direction of the wind. Under the combined action of maximum wind-load and aerial-pull, a heel of 1 per cent. is permissible, that is, the top of the mast may move 8 feet horizontally.

Fig. 10.



In order to calculate the stay tensions and mast stresses under maximum load, the mast has been treated as a continuous girder, the points of support to the stays being regarded as lying on the same straight line. By the application of the theorem of three moments, the bending moment at the points of support, and thus the horizontal reactions and the horizontal components of the stay tensions, were obtained. From these values of stay tension the correct initial stay tensions were calculated. With the masts in a vertical position, the initial stay tensions are so set as to ensure that, when the mast heels over 1 per cent., the horizontal components of the stay tensions will equal the reactions as calculated by the application of the theorem of three moments. *Fig. 9* shows the bending-moment and shear diagrams calculated by the foregoing methods. The bending-moment about the pivoted base is, of course, zero. In *Fig. 10* $M_1, M_2, \ldots M_5$ represent the mast bending-moments at the points of stay attachment, and M_0 the bending-moment at the base of the mast. The vertical distances between stay attachments are denoted by $l_1, l_2, \ldots l_5$, whilst w, w_1, $w_2, \ldots w_5$ are the horizontal wind loads per linear foot of the masts; P is the horizontal pull due to the aerial. The sections of mast between stay points are uniform, and the wind load per linear foot is also considered as uniform.

Then

$$M_4 l_4 + 2M_5 (l_4 + l_5) + M_0 l_5 = -\frac{1}{4} (w_4 l_4^3 + w_5 l_5^3)$$

$$M_3 l_3 + 2M_4 (l_3 + l_4) + M_5 l_4 = -\frac{1}{4} (w_3 l_3^3 + w_4 l_4^3)$$

$$\ldots\ldots\ldots = \ldots\ldots$$

$$M_1 l_1 + 2M_2 (l_1 + l_2) + M_3 l_2 = -\frac{1}{4} (w_1 l_1^3 + w_2 l_2^3)$$

Since M_0 is zero, and since the pull, P, on the top of the mast is known, $M_1 = Pl + 0{\cdot}5\, wl^2$ can be calculated. Hence all the bending-moments can be ascertained. From these values of bending-moment and the horizontal loads on the various mast sections, the shear at each side of the stay attachment, and thus the horizontal reaction, can be obtained readily by the usual methods. A slight correction of

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the horizontal reaction at each stay-point attachment is necessary, due to wind load on stays. It is essential to remember that the total horizontal reactions thus obtained are the values when the mast has heeled over to its final position. In consequence, the stay-ropes are almost taut, and the actual tension in each stay is thus equal to

$$\frac{\text{horizontal reaction} \times \text{length of stay}}{\text{horizontal distance of anchor-block from base of mast}}.$$

Table I gives the horizontal reactions and the actual maximum tensions in the stays at the points of attachment to the masts, due to the combined action of aerial-pull, wind load on mast, and wind load on stays. This Table also gives the correct original tensions necessary in the stay-ropes at the anchorages in order that the values shown in column 6 shall be obtained when the centre-line of the mast heels over 1 per cent. As already mentioned, the actual

TABLE I.

Stay Number.	Vertical Height of Point of Attachment of Stay to Mast.	Horizontal Distance from Base of Mast to Anchor-block.	Actual Cross-Sectional Area of Stay.	Horizontal Stay-Tension Due to Combined Action of Wind on Mast and Stays and Load on Aerial.	Actual Maximum Tension at Top of Stay.	Correct Original Tension at Bottom of Stay.
	Feet.	Feet.	Sq. Ins.	Tons.	Tons.	Tons.
Top V	810	600	1·94	21·6	36·7	4·7
IV	652	600	1·44	22·1	32·7	4·3
III	490	400	1·44	22·0	34·8	2·3
II	328	400	1·44	23·7	30 5	2·3
I	166	200	1·94	29·0	37·6	2·0
Base	..	..	..	8·6	..	..

tensions in the stays at the anchorages can be ascertained by means of the pump and gauge in the stay insulator frame. A check upon the tensions was also made from the angle between the horizontal and the direction of the stay at the anchorage, and again by measuring the sag of the stay by means of a theodolite.

In calculating the original stay tensions, each stay was regarded as a section of a catenary (B C D in *Fig. 11*). B is the point of attachment to the mast and A the anchorage of the stay. It can be shown that if L denotes the length of stay, K the vertical height

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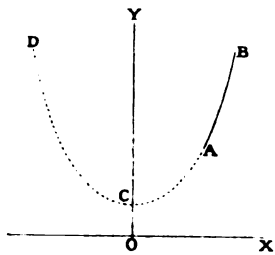
of B above A, h the horizontal distance of B from A, and c the parameter,

$$L^2 - K^2 = c^2 \left[e^{h/2c} - e^{-h/2c}\right]^2$$

Therefore $$L^2 = c^2 \left[e^{h/c} - 2 + e^{-h/c}\right] + K^2,$$

and $$L = \sqrt{2\, c^2 \left[\cosh (h/c) - 1\right] + K^2}$$

Fig. 11.



Other formulas employed in calculations of stay tension are :—

$$L = \sqrt{\left(\frac{24\, c^2\, h + h^3}{24\, c^2}\right)^2 + K^2} \text{ approximately}$$

$$X_A = c \log \frac{K + L}{c\,(e^{h/c} - 1)}$$

$$Y_A = c \cosh \frac{X_A}{c},$$

$$Y_B = c \cosh \frac{X_A + h}{c}.$$

where X_A is the horizontal co-ordinate of A referred to the axis of the catenary,

and Y_A and Y_B are the vertical co-ordinates of A and B referred to the directrix.

The method of applying these formulas is illustrated in another Paper by one of the Authors.[1]

The vertical components of the stay tensions produce compressive stresses in the mast posts, the effect, of course, being cumulative from top to bottom. In arriving at the maximum compressive stress in any post, the maximum stress due to bending was added to that due to the vertical component of the stay tensions and the mass of the superstructure. The value so obtained was that which complied

[1] T. Walmsley, "Length, Inside, and Sag of Stay-Ropes." ("Inst. C.E. Selected Engineering Paper No. 19.") 1924.

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with the stipulation that the maximum stress should not exceed 14,000 lbs. per square inch, and should not be greater than the value obtained from the expression $18{,}000 - 80l/r$. Except in the short cantilever top of the masts, the combined stresses in any post are never tensile in any section, the tensile stress due to bending at any section being less than the sum of the compressive stresses due to the mass of the material and the vertical components of the stay tensions. This fact suggests that circular tube posts with suitable spigot-and-socket extremities might well be used in mast-construction.

Mast- and Stay-Insulation.—It is generally conceded that, from theoretical considerations, masts insulated from earth are preferable to the non-insulated type, the ideal mast being that which least disturbs the homogeneity of the electrical and magnetic properties of the medium between aerial and earth. Such an ideal is impossible of practical attainment. A near approach to the ideal would be to insulate a mast suitably in several places throughout its length; but considerations of cost and safety would render such a construction impracticable. Indeed, it is a matter of contention whether, by insulating the mast solely at the base, the gain in effective height of the aerial and the reduction in energy-losses in the mast are worth the extra cost and risk. Whilst theoretically the gain in effective height is unquestionable, the experience of various authorities is very contradictory. In some cases it is denied that there is any practical advantage. It is more than probable, however, that where no advantage has been gained, the essentials of mast-insulation have not been thoroughly appreciated. The requirements are not merely that the mast must be insulated from earth, but also that the capacity of the mast to earth should be reduced to a practical minimum. In other words, the masts must be raised considerably above the earth. A mast separated from earth by a few inches of porcelain or other insulating material, although electrically insulated as far as low-frequency currents are concerned, offers a ready path to earth for high-frequency currents. The problem of mast insulation is thus resolved into a search for an insulator of considerable thickness, capable of withstanding the simultaneous application of both high-frequency electrical stresses and high mechanical stresses. Many tests upon various materials were made before the design of the base of the masts was decided upon. The results of these tests may be summarized briefly as follows:—

(1) Porcelain is the most suitable material for an insulating base, although certain classes of granite have high insulating properties.

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(2) Many classes of porcelain, whilst capable of withstanding the application of high voltage at low frequency, heat and even melt under the application of high-frequency electrical stresses.

(3) Many porcelain insulators, whilst capable of withstanding high-frequency electrical stresses, fail under high mechanical compressive stresses.

(4) Only certain classes of granite and porcelain are able to withstand the simultaneous application of both high potential stresses of radio frequency and high mechanical compressive stresses.

Extensive tests made by Post Office engineers on porcelain insulators show that the best type for the purpose of mast-insulation is cylindrical in shape, commonly known as the "cheese" type. The insulator must not be too thick—a suitable thickness is 3 to 4 inches—in order that, in the process of firing, good vitrification throughout the substance of the insulator may be obtained. The best results are given by insulators having a cylindrical hole through the middle. The hole not only ensures effective firing, but also facilitates radiation of heat due to electrical energy-losses. The insulators used at Rugby have a diameter of 9 inches and a thickness of $3\frac{1}{2}$ inches. Twelve columns, each consisting of three porcelain insulators making a total height of $10\frac{1}{2}$ inches, support each mast. Additional insulation is provided by a column of granite 5 feet 6 inches cube. The construction of the base of the mast is shown in Figs. 4, Plate 7. A steel stanchion, fixed to a foundation grillage of rolled steel joists, supports the granite blocks. It is considered of some importance, in order to reduce the capacity of the mast to earth, to have the stanchion as slender as is compatible with mechanical strength. Also, careful selection of the granite is essential. Tests upon many samples of granite have shown that British granite is unsuitable as a high-frequency insulator. For this reason, the stone used at Rugby is Swedish. A typical curve of the temperature-changes in Swedish granite due to the application of high-frequency voltages, is shown in *Fig. 12*. It will be observed that the temperature rises more or less rapidly to about 210° F., according to the quality of the granite. In the case of suitable samples the temperature then drops slowly until it reaches a steady value a few degrees higher than that of the atmosphere. The inference to be drawn from this curve is that there is considerable loss of energy in the granite until the moisture has been driven out. Thereafter the energy-loss rapidly decreases. The effect of even slight traces of moisture

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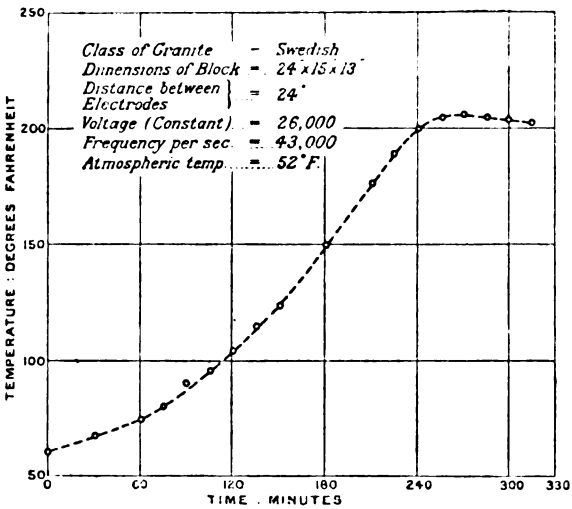
increases the energy-losses so much that at Rugby it was considered desirable to provide some effective protection of the granite bases. The possibility was considered of heating each block in a closed oven before erection, and coating it, whilst hot, with bitumastic solution. Tests on smaller samples had shown that these measures were quite effective in extracting and excluding moisture. The objection to this method in the case of the large blocks was that, in order to make a good cement bond between blocks, it would have been necessary to leave the horizontal faces uncoated, with the result that further absorption of moisture might have occurred. With the blocks grouted in position, difficulty might have been

Fig. 12.



experienced in driving this moisture out. It was therefore finally decided to erect the blocks undried and unpainted. In order to avoid a vertical layer of semi-conducting material, care was taken to place no cement between the vertical joints of the blocks. If, when current is flowing in the aerial, there are appreciable losses in the insulators, the moisture will be driven out, and the blocks may then be coated with suitable insulating solution.

Aerial.—The aerial system is formed of a series of "sausages" suspended between the masts and divided into two sections. One section is formed of the eight masts on the south of the station, the other of the four masts on the north. Each section is fed by independently insulated "trunks" on the two masts adjacent to the

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station buildings. The two isolated aerials can be utilized for simultaneous transmissions, or by internal bus-bar connections the two aerials may be linked for one transmission in full power. Initially the large aerial will be used for telegraphy and the small for telephony. The respective capacities and natural wave-lengths are as follows :—

	Capacity : Micro-microfarads.	Wave-length : Metres.
Eight-mast aerial	31,700	8300
Four- „ „	22,700	6150

Each of the sausages of the eight-mast aerial is formed of eight 7/14 silicon-bronze cables mounted on 12-foot diameter spiders of high-tensile tubular steel, supported on a central triatic of 2-inch steel rope. On the four-mast aerial the sausages are identical, but " copper-clad " wire is used instead of bronze. At each mast the triatic is connected to an insulator attached to the aerial-halyard. The advantage of this type of aerial is that a minimum dip of wires, and thus a greater height of aerial, is possible. With the flat-topped type of aerial, supported on steel cables suspended between masts, the dip of the supporting wires must be added to those of the aerial-wires, whilst in the case of the sausage type, the dip of the aerial as a whole is practically the same as that of the central cable.

There appears to be little to choose between the sausage aerial and a flat-topped aerial supported upon horizontal projecting arms, each type possessing certain advantages. *Fig. 13* enables a comparison of the capacity of the two types to be made. It will be observed that wires suspended from horizontal arms having a pitch of about 34 feet would have the same capacity as a twelve-wire sausage 12 feet in diameter. Also, little is to be gained by increasing the number of wires in the horizontal aerial from eight to twelve.

If, however, the pitch is increased to 110 feet, the twelve-wire flat-topped aerial has 21 per cent. greater capacity than a twelve-wire 12-foot diameter sausage aerial, whilst the eight-wire flat-topped type has 19·4 per cent. extra capacity over the eight-wire 12-foot diameter sausage. Thus the flat-topped aerial shows a marked advantage as regards capacity. On the other hand, for the same total heights of masts, the advantage, both as regards average height of aerial and cost of masts, is on the side of the sausage system. In other words, the cost per unit of capacity, elevated to the same height above ground, does not differ greatly in the two systems.

The lead up to the aerial consists of eight 7/14 silicon-bronze wires, arranged on spreaders 6 inches in diameter. The object of this small diameter is to reduce the capacity where it is not required, consistent with a low value of high-frequency resistance.

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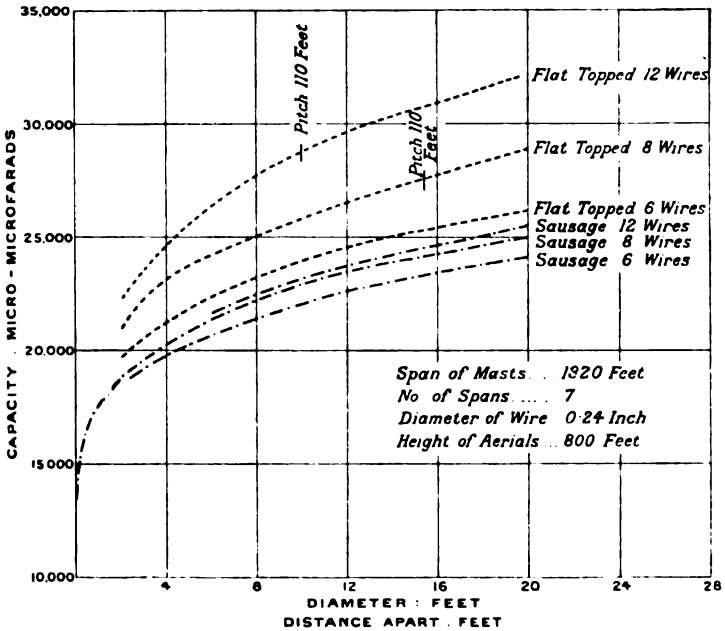
The aerial insulators consist of hollow porcelain tubes of 6 inches external diameter and 3½ inches internal diameter. The complete insulator comprises four such tubes, fixed as shown in Fig. 14, Plate 8. Guard-rings are used at each end of the insulators. Similar porcelain tubes are used for taking the horizontal pull of the down lead, six tubes being arranged in star, two tubes in series forming each leg of the star. These insulators are tested to a proof load of 20 tons and to a potential of 200,000 volts at a frequency of 50,000 cycles per second.

Fig. 13.



Earth.—The earth system for the eight-mast aerial is formed of copper wire, weighing 100 lbs. per mile, buried about 6 inches below the surface and covering the vertical projection of the aerial, extending to a distance of about 820 feet—the height of the masts—on either side of the centre-line. The general arrangement of the earth system is indicated in Fig. 15, Plate 8. The wires were laid by a specially-designed plough which carried a reel of wire and fed the wire into the narrow furrow made by the blade of the plough at one operation.

A counterpoise or raised capacity has been adopted instead of an earth for the smaller aerial system. This comprises a series of copper-

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clad wires extending to the end of the aerial and approximately parallel to it, supported on 30-foot poles, forming a screen about 1,450 feet wide. Thirty-one wires are used in each of the two arms of the counterpoise, the spacing ranging from 80 feet to 40 feet from the edges to the centre. The wires are grouped in sections, insulated from each other, and running from the open ends of the screen back to the transmission building, in order to minimize cross currents and to equalize the main oscillating currents in the wires.

Transmitting Plant.—Western Electric valves of the metal-and-glass water-cooled type are used for transmission, their rating being as follows :—

Anode potential	volts	10,000
Power output	kilowatts	10
Anode dissipation	„	10
Filament current	amperes	41·5
Filament emission	„	7 to 8

The water circulation required is 2 gallons per minute per valve, and the temperature-range permissible is 50° C. above normal temperature.

The valves are arranged in panels of eighteen valves each. Five complete panels are installed, and by means of suitable switches any panel can be separately connected to the main bus-bars. Three panels, comprising fifty-four valves, are required to generate 500 kilowatts of high-frequency power in the aerial circuit.

Filament Supply.—One filament transformer is provided for each valve-panel, and is supplied with three-phase alternating current at 100 cycles frequency by special motor generators. The regulator for the latter is fitted in the valve-room, and each filament can be separately controlled by means of a rheostat.

The valve power-units are excited by means of a valve oscillator-unit comprising three water-cooled valves excited in turn by a 600-watt glass valve which may be arranged in a self-oscillating circuit. The grids of the valves normally are given a negative bias by means of motor generators; master control of oscillations is provided by a tuning-fork oscillating set.

An intermediate circuit of Dubilier condensers, arranged in two bands of ten of 0·025 microfarad capacity and ten of 0·1 microfarad capacity, in conjunction with a stranded-wire coil of 500 microhenries inductance, is provided, to ensure that the emission of harmonics is reduced to a minimum. The total weight of the condenser is 2½ tons.

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The inductance coil is formed of three hexagonal spiders of approximately 14-foot diagonals, each wound with four turns of cable consisting of 6,561 strands of 36 S.W.G. enamelled wire, in bunches of 81 strands, provided with silk coverings. The spiders are adjustable relatively to one another, the spacing ranging from 1 to 3 feet. The circuit is designed to give a wave-length of 18,000 metres.

The aerial-tuning inductance is arranged similarly to the intermediate-circuit coil, and consists of five spiders of 15-foot 6-inch diagonals wound with eight turns of similar cable per spider.

An overload relay is fitted in the anode lead of each valve, to ensure that the failure of any valve will cut off the main supply by the operation of a direct-current high-tension switch. Each valve-panel is fitted with a water security control which cuts off the main current-supply in the event of the water-supply falling below the safe minimum allowable. An interlocking device is fitted, to prevent access to any valve-panel while it is alive.

In addition to the telegraphy plant described above, a Western Electric 200-kilowatt radio transmitter is being installed for two-way telephony with a similar station in the United States.

Power-Supply.—Three-phase alternating current at 50 cycles, 12,000 volts, is supplied by duplicate mains, to ensure continuity.

The main d.-c. supply to the valves is provided by means of British Thomson-Houston 500-kilowatt, 6,000-volt motor-generator sets. Each set consists of one synchronous motor driving two 3,000-volt single-commutator generators joined in series, and two pedestal exciters. The five machines are mounted on one bed-plate supported on groups of porcelain insulators. The motors, which are self-synchronizing, are rated at 640 kva., 2,200 volts, 0·9 leading power-factor, speed 750 revolutions per minute.

In order to avoid the use of insulated couplings, each motor is supplied through a transformer specially insulated to withstand 50,000 volts between primary and secondary windings. The mid-point between the two generators is connected to the bed-plate through a trip relay, and the neutral point in the motor is also connected to the frame.

The exciter for the synchronous-motor field is rated at 8·5 kilowatts, 65 volts, and the other for the shunt of the d.-c. generators at 8·5 kilowatts, 110 volts.

Three complete motor generator sets are installed, being arranged for series connection to give 12,000 or 18,000 volts continuous current as required. Each set is provided with two high-speed circuit-breakers, which throw a limiting resistance into circuit on

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a short circuit occurring. The operation of any one of the circuit-breakers trips the field circuit of the generator. In addition, normal overloads operate a relay actuated by a shunt in the main negative lead and trip the field circuit of the high-tension generators. The main alternating-current oil switches are provided with overload trips with time-limit fuse. To start the motors, the primary side of the machine transformers is switched directly on the 12,000-volt mains in star. By means of a change-over switch, the connections are then changed to delta.

The 100-cycle-frequency supply for the valve-filament current is provided by frequency-converters. Each set consists of one three-phase, 416-volt, 50-cycle synchronous motor coupled direct to a three-phase, 416-volt, 100-cycle, 200-kva. alternator.

Continuous current at 240 volts is provided for controls, excitation for motors of frequency-converters, etc. Two motor generator sets, each consisting of one three-phase, 416-volt, 50-cycle induction motor coupled direct to a 25-kilowatt, 240-volt, shunt-wound generator and battery-booster, are installed for this supply.

A 200-ampere-hour battery is also provided for stand-by lighting and control purposes.

Power for mast hoists, lighting, and pumps for cooling water for valves is taken direct from the main a.-c. bus-bars at 416 volts by transformers.

Erection.

In the course of the construction of the radio station many interesting problems of erection arose. It is proposed to dwell briefly upon some of the problems under the headings of masts, stay-ropes, and earth and aerial systems.

Masts.—The steel triangulated base of each mast, weighing about 3 tons and having a length of 12 feet, was built in the shops and transported by rail and road to the radio station, where it was lifted into position by means of a Scotch derrick. Originally the tripod was supported in position by stout wooden props, extending from the concrete foundation to projecting angles bolted to the sides of the tripod. It was found, however, that the props were unnecessary, and that temporary stays provided a quicker and less expensive means of support. A wooden derrick was then placed inside the tripod, and lengths of post were raised one by one and braced together round the derrick. As each section of mast was completed, the derrick was raised to a higher level, and an additional section was then built round. The economical limit to erection by this method was about 90 feet. When a mast had been completed to this height,

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the erecting-cage proper (Fig. 16, Plate 8) was fixed inside. This consists essentially of a vertical steel tube 1 foot 6 inches in diameter and 48 feet long. Six horizontal platforms, P, pitched 9 feet apart —the same distance as the horizontal mast-bracings—are bolted to the central tube. The upper extremity of the tube terminates in a cathead, the horizontal arm of which is capable of rotation in a horizontal plane. The hoisting-rope passes from the drum of the ground electric winch up the middle of the tube and over the various sheaves of the cathead. After a post or bracing had been raised to its approximate final position, exact adjustment was made by hand at the cathead, by sliding the sheave A along the horizontal arm of the cathead and by raising or lowering other sheaves. These movements were accomplished by rotating horizontal and vertical screws, to which the sheave-brackets were attached. To raise the erecting-cage three hand winches at ground-level, attached to the heel ropes H of the blocks B were used. When the cage had been raised sufficiently, the bolts C were shot into position by hand levers, so as to support the mass of the cage upon the horizontal straps of the posts. During the course of erection temporary stays were employed, permanent stays being fixed as soon as each permanent stay-point was reached. When the last section of mast was completed, the cathead was first removed and a stout wooden pole was lashed to one of the posts. By means of the ground electric winch and blocks fixed to the temporary pole, the top mast, aerial-arm, and sheaves were raised to position and fixed. The erecting-cage was then lowered to the bottom of the mast and removed.

The method of erection outlined above worked very satisfactorily. Modifications of it were tried, but were abandoned as being less efficient. The experiment was made of building about 70 or 80 feet of the lower end of the mast on the ground, and lifting it into position by means of steel sheer legs. The time required to fix, raise, and carry the sheers neutralized any advantage there might otherwise have been. Again, in order to avoid a large number of temporary stays, a steel frame was built round the mast, just clearing the posts. Stays were attached to the frame, which, as each small section of the mast was erected, was raised by means of blocks, the heel ropes being attached to winch drums fixed on the ground. At the same time the stays attached to the frame were let out, a good tension being maintained throughout the operation. As each permanent stay-point was reached, the permanent stays were fastened to the mast, and the frame was gradually raised to the next stay-point. The object of this method of holding the mast during erection was that, by allowing the stay-frame to follow closely in

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the wake of the erection-cage, no great length of unstayed mast would at any time overhang the frame, and the necessity of a large number of small temporary stays between permanent stay-points would thereby be obviated. The idea, however, was abandoned for two reasons: firstly on account of the loss of time in raising the frame, and secondly owing to the danger of excessive eccentric loading of the mast, due to the possibility of the tension in one of the frame stays exceeding considerably the tensions in the others.

As previously stated, the insulators under each mast consist of twelve columns, each of three porcelain blocks 3½ inches thick. The masts were built upon wooden blocks 1 foot high, which were thus a little more than 1 inch higher than the porcelain columns. Several methods of fixing the insulators in position were investigated and tried, but the most successful was the following: The three insulators comprising each column were cemented together by a very thin layer of Portland cement and allowed to set. Three hydraulic jacks were then placed under the mast, and the load was taken off the wooden blocks. Screw jacks were also inserted, in case of failure of the hydraulic jacks, but were adjusted so as to take no load. The wooden blocks were then removed, and the porcelain columns were inserted, thin layers of cement having just previously been placed below and above the insulators. The mast was then gradually lowered into position, the jacks being left until the cement had set. The presence of the cement permitted fairly even distribution of load, and when experience had been gained in fixing the insulators, very few breakages occurred.

Stay-Ropes.—As previously explained, all permanent mast-stays are composed of straight wire. Fig. 17, Plate 8, shows diagrammatically the plant used in fabricating these ropes. Extending the full length of the rope-walk is an endless rope, used to draw out the wire from the two coils shown, by means of a jockey clamped to the endless rope. Wires were also drawn out upon the return journey of the jockey. At each end of the walk the wires were threaded through holes in the plates A and A_1. Originally one end of each wire was bent over behind the plate, whilst the other end was attached to a tension screw fixed in front of the farther plate. This, however, was found to be unnecessary, and much time was saved by substituting for the tension screws steel wedges driven into the holes of the plate. After all the wires had been drawn out, the next step was to bind them tightly together, the essentials being parallelism of the wires and avoidance of twist or bunching. The plates B and D, and the clamp C, were rigidly connected together by means of steel bars. The two plates were perforated, the centres of the holes, which

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were slightly larger than the wires, being arranged in concentric circles. As the wires were drawn out, they were threaded through the holes in B and D before being passed through the holes in the fixed plate A. When the full number of wires had been drawn out, the circular clamp C was fixed, and a steel rope was attached to the bars E. By means of a hand winch fixed near the plate A, the appliance was drawn along the stay, thus closing the wires uniformly together. At the same time, a binding of galvanized mild-steel wire was served round the stay as shown at F. This binding was arranged in a spiral of about 1 inch pitch, and a close binding, not less than 3 inches long, was wound round at distances of not less than 3 feet. The ropes, both before and after being drawn together, were painted with a bitumastic solution. Many trials were made of modifications of the above form of binding, but with indifferent results. A long spiral binding, with strong wire clips instead of the closed binding, was tried, but with disappointing results. Close bindings at distances greater than 3 feet were insufficient to prevent bunching during the erection of the ropes, which was one of the greatest troubles experienced. This usually appeared whilst the ropes were being handled, the only preventative apparently being a sufficiency, both in number and in length, of close bindings. A significant fact is that bunching was almost entirely absent from ropes of smaller diameter, a fact which supports the theory that the trouble was caused by bending the ropes whilst conveying and hoisting them into position. The wires in the convex side of the rope at a bend were under greater tensile stress than those in the concave side; thus, unless the bindings were sufficiently tight and numerous, relative slipping of the wires occurred, resulting in slackness at certain parts of the rope, chiefly at both ends.

Earth and Aerial.—For the purpose of laying down the earth system the machine shown in Figs. 18, Plate 8, was designed. It consists essentially of a V-shaped steel plate having three rollers fixed across the faces of the V. The plate is bolted to two steel channels which act as skids. A frame to support a drum of wire is also fixed to the rear of the channels, and guiding handles are provided. The appliance was drawn by a tractor. The sharp edge of the V cuts the earth, which is thus parted to form a narrow trench about $\frac{3}{4}$ inch wide and 6 to 8 inches deep. As the plough moves forward, the wire is unreeled and laid in the bottom of the trench. The weight of the skids, pressing near the sides of the trench, causes the top of the opening partially to close, leaving the wire buried. About 80 miles of wire were laid by the plough at a fraction of the cost that would have been incurred had hand labour been employed.

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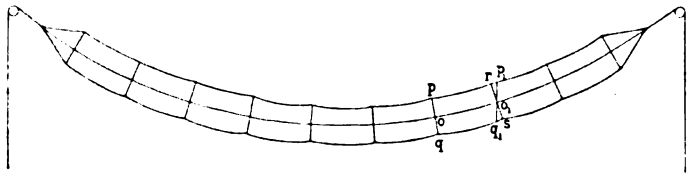
The erection of the sausage aerial presented features of interest. For electrical reasons large dips in individual wires are undesirable, owing to the variation of capacity of the aerial due to swaying of loose wires. The effect of this variation is twofold. Firstly, variation in aerial current is occasioned; secondly, sudden voltage-rises at the anodes of the transmission-valves are caused. It was thus imperative that the wires of the aerial, when suspended, should have the minimum dip between spreaders compatible with mechanical safety. This requirement could only be met by careful adjustment of the lengths of individual wires when the aerial was being built on the ground. Now, on account of the long span between the supporting masts, the dip of the central steel cable was necessarily large, being about 70 feet. The circular spreaders, 12 feet in diameter, were securely clamped at right angles to the central cable. As a result of the large dip of the central supporting cable and the large diameter of the spreaders, the lower aerial wires must be appreciably longer than the central cable, whilst the upper aerial wires must be

Fig. 19.



shorter. Of necessity the aerial had to be built on the ground, the central cable being supported upon trestles 7 feet high spaced 140 feet apart, and the spreaders fixed parallel to each other. Referring to *Fig. 19*, illustrating the aerial in its final suspended position, regarding pq and rs as adjacent spreaders, and drawing $p_1 q_1$ parallel to pq, the angle $q_1 o_1 s$ represents the angle between adjacent spreaders, and also the difference in the angle of slope of the central cable at o and o_1. Assuming the curve of the cable to be a parabola, obeying the law $y = kx^2$, the difference in these angles of slope $= \tan^{-1} 2kx_2 - \tan^{-1} 2kx_1$. Thus the lengths $q_1 s$ and rp_1 equal approximately $o_1 s \tan(\tan^{-1} 2kx_2 - \tan^{-1} 2kx_1)$. Actually this extra length was not more than 0·2 per cent. of the span between spreaders, but had all the wires of the cage been made of equal length and strained originally to 50 lbs. per wire, this extra length would have produced a dip in the upper wires of about 7 feet in a span of 140 feet and would have considerably overstressed the lower wires.

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The erection of the aerials necessitated a considerable amount of ingenuity, the greatest difficulty being to avoid the mast-stays. *Fig. 20* shows isometrically the arrangement of the sausages. At

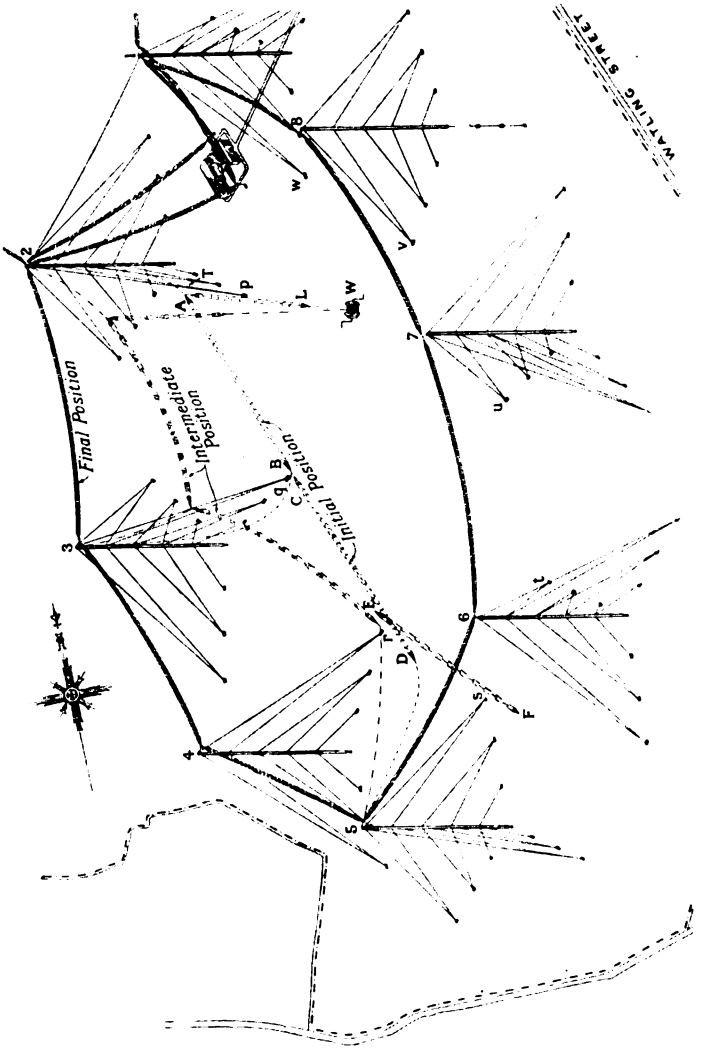

Fig. 20.

each mast one aerial-halyard, terminating in a built-up porcelain insulator, is attached to the junction of two sausages. In the final position of the aerial, the junction of the two sausages and the aerial-

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halyard is about 20 feet away from the mast. Of necessity, therefore, the electrical joints between the wires of the sausages, and also the attachment to the aerial-halyard had to be made on the ground. To avoid the stays, the aerial was built within the area enclosed by the lines joining the anchor-blocks p, q, r, s, t, u, v, and w. From the point of view of erection, it would have been simpler to suspend each span of sausage directly between adjacent masts, passing electrical connections over the top of each mast. Such a method, however, was open to the objection that the number of insulators required would have been increased considerably and the losses in the masts, due to the close proximity of electrical connections between sausages, would also have been higher. Some conception of the method of erection employed will be gathered from *Fig. 20.* AB, CD, and EF are three 440-yard lengths of aerial at ground-level. The steel central rope of a sausage rested upon the erecting trestles. Soldered joints between the individual wires of the eight 7/14-gauge silicon-bronze cables were made at BC, and the thimbles of the central steel cable were shackled to the terminating insulator of the aerial-halyard of mast No. 3. Similarly, the ends A and T of the sausage AB and the leading-in trunk TL were joined electrically and shackled to the terminating insulator of mast-halyard No. 2. The other end of the leading-down trunk was attached to the rope of a winch W, and the aerial-halyard of mast No. 5 to the end D of the sausage CD. The halyards of masts Nos. 2 and 5 were then raised until the whole of the two aerial spans AB and CD were lifted from the ground. Thus, between masts Nos. 2 and 5 there was a span of aerial and cable of nearly $\frac{3}{4}$ mile, weighing approximately 5 tons. The next step was to manipulate the halyards of the masts and the rope of the winch W, so as to swing the two spans of aerial into the intermediate position shown in *Fig. 20.* The halyards of masts Nos. 2 and 3 were raised, and the halyard of mast No. 5 lowered, until the end D could be shackled to the end E of the span EF. Employing methods very similar to those described above, the span EF was then slung forward until the end F was near to the anchor-blocks. Thus span by span the aerial was picked up and slued into position.

Tests.

In addition to the usual tests for the quality of material, tests on structures, machines, and insulators were made to determine whether the requirements of the specifications had been met.

Masts.—A horizontal load of 10 tons was applied to the upper extremity of the masts by hoisting a steel cable between two of them.

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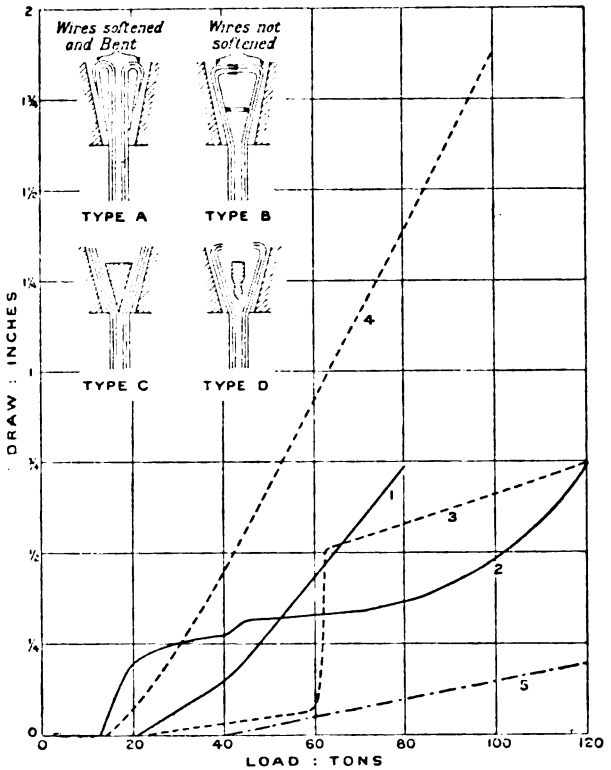
The dip, which was measured by means of a theodolite, was adjusted to the correct value to produce the desired horizontal tension, and the deflection of the masts from the vertical was determined. It was 3 feet 6 inches at the top. This deflection is comparable with the 8 feet maximum deflection allowed for with maximum wind-pressure and aerial-pull. Whilst this tension was maintained,

Fig. 21.



adjustment of the slipping-gear was made so that the shoes were on the point of slipping.

Stay-Sockets.—Many tests on the socketed ends of stays were made for the purpose of ascertaining the most suitable form, an Olsen machine capable of a maximum pull of 125 tons being used for the purpose. *Fig. 21* illustrates four types of sockets and gives graphs of the tests upon the various types. The graphs show the amount of "draw" of the rope from its socket under increasing

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loads. In socket A the wires were heated and bent over as shown. The weakness of this type lies in the fact that the bent wires shield the central core from the molten metal poured into the socket. Thus the outer wires are firmly held by the metal whilst the pull on the rope is concentrated almost entirely on the softened bent extremity, with the result that the wires break at the bend. With the type B socket much better results were obtained. The wires were fanned out, a soft iron washer being used for that purpose. By this means the molten metal was able to come into intimate contact with a maximum number of wires, with the result that the socketed end of the rope was as strong as the rope itself. In type C socket a steel wedge was driven hard between the wires before the metal was run in. Curve No. 3 in *Fig. 21* illustrates a common defect of this type. When the load was approximately 60 tons, the rope gave in the socket about ½ inch. The type of socket finally adopted as being the easiest to make and productive of the best results is type D. The washer of type B is replaced by a small plug, which is used for the purpose of spreading the wires. It is so shaped as to allow the molten Babbitt metal readily to run round the wires in the socket, and thus to permit of a maximum adherence. To obviate the sudden yielding frequently experienced with the wedge type of socket, a good tension was placed upon the rope before the metal was poured into the socket.

Table II summarizes the results of the tests.

TABLE II.

Curve No.	Circumference of Rope.	Type of Socket.	Remarks.
	Inches.		
1	5	A	Wires broke in socket at 85 tons. Rope withdrew entirely at 88·4 tons.
2	5	B	Bend in rope, hence only 25 per cent. of wires in tension at start. Did not withdraw from socket at 125 tons.
3	6	C	Rope did not withdraw at 125 tons.
4	6	A	Wires started breaking in socket at 100 tons. Rope withdrew entirely at 105 tons.
5	5	D	Rope did not withdraw at 125 tons.

Curve No. 2 is interesting as showing the effect of bunching in the wires of a stay-rope. Owing to a large bend in the test-piece of rope, the bunching was exceptionally noticeable at the beginning of the test, no more than 25 per cent. of the wires being in tension. In consequence, the rate of draw from the socket was rapid until the load reached about 20 tons, when additional wires became

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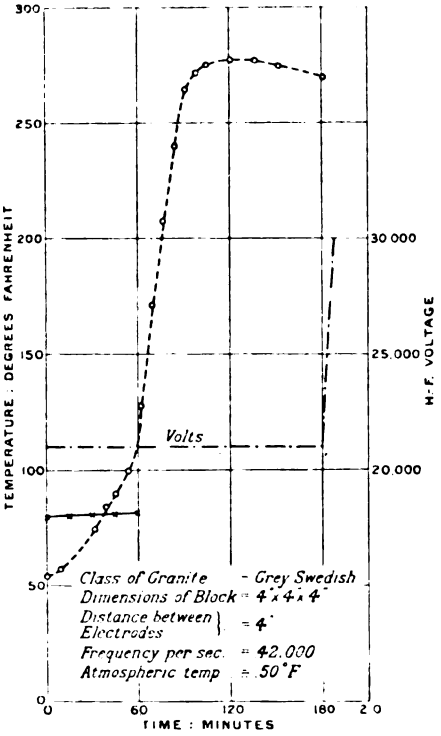
engaged. Thereafter, until 40 tons load was reached, the draw increased very slightly; but at 40 tons there was again a marked jump. More wires were then engaged, until at 100 tons nearly all of them were under tension.

Tests upon the adhesive force of the Babbitt metal used in the sockets were made by dipping 1- and 2-inch lengths of No. 10 S.W.G. galvanized wires in molten Babbitt metal and allowing to cool. The results were very uniform, the wires pulling from

Fig. 22.



the metal under a load of 13 cwt. per inch length of wire immersed. The metal used (85 per cent. lead and 15 per cent. tin and antimony), had a melting-point of 460° F.

Insulators.—As previously stated, tests were made upon many materials to ascertain their suitability to withstand the application of high voltages of radio frequency. Table III summarizes some of the results upon granite, which may be taken as fairly typical of many samples tested. The marked superiority of Norwegian

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granite over British is at once apparent. Swedish granite has properties equally as good as Norwegian. From the electrical standpoint, Aberdeen granite is the best of the British granites. Whinstone was found to be totally unsuitable as an insulator. The application of only 1,000 volts at 43,000 cycles per second caused a sample 4-inch cube to attain a temperature of 417° F. in 3 minutes, and the material commenced to disintegrate.

TABLE III.—TESTS ON GRANITE.

Frequency of Supply in all cases 43,000 cycles per second.

Origin of Sample.	Condition of Sample.	Dimensions (First Dimensions = Distances Between Electrodes.)	H.-F. Voltage applied.	Duration of Test.	Temp.-Rise.	Remarks.
		Centimetres	R.M.S. Volts.		Deg. F.	
Norway .	Dry	10×9½×5	25,000	7 hours	70	Withstood voltage without breakdown. Failed mechanically by crushing at 3·1 tons per square inch
Norway . (Coarse grain)	Dry	,,	21,000	5 ,,	58	Withstood voltage without breakdown
Norway . (Fine grain)	Dry	,,	21,000	5 ,,	48	,,
Norway . (Coarse grain)	Wet	,,	3,250	3 mins.	100	Sample submerged in water for 30 minutes before test
Norway (Fine grain)	Wet	,,	3,250	3 mins.	73	,,
Britain .	Dry	11×8×3	25,000	Few secs.	95	Arcked over immediately
Britain, Cornwall	Dry	15×15×8	17,000	10 mins.	90	Test discontinued
Britain, Aberdeen	Dry	10×9½×5	21,000	5 hours	102	

Reference has already been made to *Fig. 12*, showing the test results on a sample of Swedish granite.

Fig. 22 gives the results of a test upon a 4-inch cube of grey Swedish granite, and illustrates in a very marked manner the effect of moisture. The test is admittedly rough, but is, nevertheless, instructive. As in the test shown in *Fig. 12*, the temperature of the sample rose rapidly to a maximum value, and thereafter slowly

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fell, even though 21,000 volts r.m.s. were maintained across two opposite faces of the cube. While still in a heated condition the sample successfully withstood a flashover voltage of 30,000 volts for 10 minutes. The granite was then allowed to cool, after which a second test was made. The temperature-rise was negligible. These tests are typical of many others upon Swedish and Norwegian granites.

Staffordshire bricks, when subjected to the application of radio-frequency high voltages, exhibited similar characteristics to those of

TABLE IV.—MECHANICAL TESTS ON PORCELAIN INSULATORS.

Origin of Sample.	Type and Approximate Dimensions.	Area.	Max. Load Applied.	Max. Stress.	Remarks.
		Sq. In.	Tons.	Tons per Sq. In.	
Denmark	Cylindrical $7\frac{1}{4}$ in. diam., 3 in. thick. Central hole 2 in. diam.	38·14	490	12·8	Single insulator. No signs of failure at 490 tons load
Denmark	,,	,,	340	8·9	Column of 3 insulators. A piece broke off middle insulator at 340 tons
Britain .	Cylindrical $8\frac{1}{4}$ in. diam., $3\frac{5}{8}$ in. thick. Central hole 2 in. diam.	53·6	240	4·5	Column of 3 insulators. Top insulator cracked at 75·9 tons. Further cracking heard at 198·4 tons. Load raised to 240 tons, then removed when all three insulators were found cracked
Britain .	,,	,,	300	5·6	Single insulator cracked at 33·6 tons, and continued to crack whilst load was raised to 300 tons

granite. They showed the same rise to a maximum temperature, and thereafter no increase or else a gradual fall. Their behaviour after immersion in water was superior to that of granite, wire-cut bricks being inferior to moulded.

The problem presented in selecting suitable porcelain insulators for supports immediately below the masts was not so much to find porcelain capable of withstanding high pressure-gradients without puncture, as to obtain insulators that would not heat under the application of radio-frequency voltages or break under high compressive loads. Porcelain cylindrical insulators a few inches thick

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could readily be obtained to withstand an average potential gradient of 10,000 r.m.s. volts without puncture, but none of the original British insulators submitted for test would comply with the combined flash, heating, and mechanical requirements. Happily, this state of affairs has now been remedied.

The specification required that the insulators were to be tested to three times the calculated maximum mechanical stress to which they would be subjected, and 5 per cent. of the insulators were to be tested to destruction, with the proviso that should any fail at less than six times the calculated maximum stress, a further 5 per cent. were to be tested to destruction. As the maximum calculated downward thrust upon the mast is 400 tons, the specified requirements are severe. Owing, however, to the difficulty of ensuring uniformly-distributed stress over the insulators, and the lack of resilience in porcelain, the requirements were not unreasonable.

As illustrating the difference between the British and the Danish sample insulators submitted for test, Table IV has been prepared. The marked superiority of the Danish insulators is apparent.

Regarding the electrical tests, every insulator before acceptance was tested by the application of about 15,000 volts at 50,000 cycles per second frequency. Any insulator showing more than a few degrees rise above atmosphere was rejected. The heating-curves of defective insulators are usually similar in form to those of granite, flattening out at temperatures between 200° and 300° F. measured externally. In a few cases, however, insulators actually became molten.

The Authors desire to express their thanks to the Engineer-in-Chief to the British Post Office for permission to publish this Paper, and their appreciation of the courtesy of Messrs. Head, Wrightson and Company, Ltd., the contractors of the mast, in supplying much useful information.

The Paper is accompanied by eight photographs, fourteen prints, and five diagrams, from some of which Plates 7 and 8 and the Figures in the text have been prepared.

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Discussion.

Dr. Eccles. Dr. W. H. ECCLES remarked that, as Chairman of the small Committee which designed the station, he was glad to take the first public opportunity of felicitating the erecting engineers on their success. The credit due to them was all the greater because the designing Committee had insisted upon many new departures from orthodox practice being introduced into the station. In the first place, as was described in the Paper, the masts were insulated; and in all other countries where insulated masts had been tried the practice was usually condemned. After making the fullest inquiries, the Committee had found that the signals were stated to be better when the insulated masts were connected to earth. In spite of that, the Committee had believed that radiation must be better if the masts were thoroughly insulated and placed high above the ground. The erecting engineers had had to meet many difficulties in carrying out that task, but they had managed it, and the results to date showed that the radiation was improved, and that, besides, the insulation of the whole aerial could be better maintained with the arrangement that had been adopted. Again, there was a movement in favour of lower masts at the time the station was under construction. It was alleged that masts 400 feet high were amply sufficient—in fact, that by appropriate methods they could be made to do better service than masts exceeding 800 feet in height. The designers had thought differently, and had insisted on having high masts; and that again had been justified by the result. It had meant many problems of detail falling upon the erecting engineers, because, he believed, high masts had not been erected before on such sub-structures. Again, the standard practice in the matter of aerials was to use what was called an inverted T aerial with numerous horizontal wires spread fairly equally over the whole area of the site. It had been resolved to depart from that, to locate the masts over an octagon instead of a rectangle, and to put all the wires on the edge of the site. Most of the numerous foreign engineers who had visited Rugby during the process of erection of the aerial considered that a wrong move, or, at any rate, a doubtful one. Again he thought the designers had been right in adopting that very unusual method. But it had thrown extra strain on the men on the site, and the planning of the mode of erecting the aerial, its construction, and the actual hoisting had fallen upon

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the Authors. The earth system was unique in that it formed a continuous ring around the octagon, which was obviously the proper method for fitting an earth system to such an aerial. Another feature which was a great departure from previous stations was the operation of the whole station by means of thermionic valves. The complete set of valves was capable of delivering 540 kilowatts of high-frequency energy to the oscillating circuits—very much more than had been attempted in any other station. Another unique feature was that the whole of the energy supplied to the aerial was controlled by a tuning-fork. A harmonic of the tuning-fork was selected and magnified in order to produce radiation. The frequency of the tuning-fork was 2,000 per second, and the harmonic selected was eight times that frequency. Although the harmonic in the fork produced only a few microwatts, that energy, when increased by thermionic multipliers to the order of 500,000 million times, controlled the whole high-frequency energy produced. The advantage of that system, which had been developed in the Post Office research laboratories and which was not used anywhere else, was that the wave-length was absolutely constant, that was to say, as constant as the fork. A tuning-fork had a very small change of frequency with temperature; if the fork at Rugby changed 1° in temperature, the number of oscillations per second in the aerial changed from about 16,000 to 16,016, an inappreciable fraction. As it would be possible to keep the fork at a constant temperature, there need not be any change at all. The practical advantage of that was seen by contrasting it with the behaviour of almost any other station, where the frequency would be found to vary slightly: instead of being constantly, say, 18,000 per second, it went up to 18,100 or sank to 17,900. Constancy of the frequency could not be attained by the ordinary methods: the result was that every station had to be given a wider wave-length band than it really ought to be allowed. If the wave-lengths of various stations were constant, they could be put much closer together, with the result that many more stations could be worked in the same band of wave-lengths. It was not known how far that advance would affect existing wireless telegraphy; it depended on whether similar forms of frequency-control were widely adopted; but it was believed that that was the trend of advance. The radiation from the station had proved to be enormous. Measurements made in America showed that it was stronger than the Bordeaux station, which previously had been the strongest of the European stations heard in America. The power was not yet at its highest; in fact, four more masts might be utilized, the valves might be pressed to higher voltages,

Dr. Eccles.

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Dr. Eccles. and perhaps in the future the present power might be almost doubled. The station was much the strongest in the world. Rumours had been current for some time with regard to the effect of the development of short-wave stations upon such a station as Rugby. He thought the situation was analogous to the arrival of the motor-car and its effect on the railways. It had been said in some quarters that railways would become immediately obsolete. As a fact, railways fulfilled different functions from motor-cars, and both were necessary. Similarly, long-wave stations of great power and short-wave stations of small power performed different functions, but both were wanted. He wished, on behalf of the Wireless Telegraphy Commission, to express his admiration of the splendid way in which the erecting engineers had tackled their unprecedented tasks, and to emphasize the gratitude which those who had planned the station extended to the Authors of the Paper and their colleagues.

Captain Eckersley. Captain P. P. ECKERSLEY remarked that, although the station which formed the subject of the Paper rather dwarfed the Daventry broadcasting station, with which he was concerned, he thought some of the problems were identical. He would be interested to learn whether the aerial had actually been tested in sections as it was erected. He noticed that the aerial lead-up "trunks" were in the form of a 6-inch diameter sausage with eight wires, and he wondered whether it would not have been stronger mechanically, and as perfect electrically, to have had a solid conductor. He did not think the resistance would be very different, but naturally with very long waves and with high-efficiency stations the radiation resistance was not very large compared with the dead resistance. The re-designing of the aerial at Daventry, which had fallen, was being considered, and though that aerial had a very high radiation resistance, namely, 7 ohms, it was felt by the engineers that they could easily afford to use solid wire and thus ensure satisfactory mechanical strength. He had been interested to hear from Dr. Eccles that there was a great difference between the results given by insulated and by uninsulated masts, and he thought everyone must admire the courage of the Technical Committee in putting their convictions into practice. Engineers, however, should not be misled into thinking that very elaborate precautions were necessary in connection with stations with a high radiation resistance compared with their dead resistance. If the aerial, from its form, had not had to be so close to the steel masts, would it not have been possible to use an uninsulated mast? An uninsulated mast was used at Daventry, but he thought that most of the troubles incident to electrical radiation were eliminated by the fact that the original

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resistance was so high that a little change did not make much difference. Captain Eckersley.

Mr. E. H. SHAUGHNESSY remarked that, as Assistant Engineer-in-Chief of the Post Office, as a member of the Wireless Telegraphy Commission, and as the engineer in charge of the work, it gave him great pleasure to congratulate the Authors on their Paper. In co-operation with the contractors they had done excellent work in bringing to a successful issue the design and the erection of the masts. Dr. Eccles had said that inquiry had been made as to the insulation of the masts; he might have added that those concerned visited a number of foreign stations, and saw the insulated masts no longer insulated, but earthed. The determining of proper insulating material had involved a good deal of investigation before it was possible definitely to say that the materials would stand the load put upon them. It was now found that when a very large current was put into the Rugby aerial the insulators stood the test well. Every insulator, before being put in the masts, was given a rigorous high-frequency test at the station, where a steam-engine and dynamo and high-frequency generator of about 100 kilowatts had been installed for the sole purpose of testing insulators before they were put into the mast structure. He believed Dr. Eccles had made a slight mistake, with regard to the frequency, in saying that for a temperature-change of 1° C. the frequency would change 16 cycles. The station had very much closer limits of frequency than that, and he thought Dr. Eccles meant 10° C., instead of 1°. Another very important factor was that with constant-frequency transmission highly selective receiving apparatus would be used, which was free from disturbance by other stations, and reduced disturbance by atmospherics. When the frequency was variable, it was not possible to have note selectivity, and consequently a great deal of the advantage of a high-power station was lost. The Paper dealt mainly with the masts, but contained details with regard to other parts of the station. He hoped to read a Paper before the Institution of Electrical Engineers dealing mainly with the electrical side of the Rugby station, but he would take the present opportunity to mention a matter concerning the valve transmitters. It was not simply a matter of putting the valves together and turning on the power; when that was done the trouble began. As many as fifty valves of 10 kilowatts output each had never been at work together before. When a number of valves were joined in parallel, they not only worked on the required wave-length, but they also interacted with all sorts of frequencies, and not only was energy frittered away, but there was also a good chance of destroying a large percentage Mr. Shaughnessy

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Mr. Shaughnessy. of the valves. Mr. R. V. Hansford, Assoc. M. Inst. C.E., had been mainly responsible for the detailed work in getting the valves to act properly, and since the station had started work there had been no stoppage due to any trouble in connection with the valves, and the reports of the signals were good. A report had been received from Australia that day that all the Press programmes sent out at midday and midnight had been received satisfactorily. The original intention of the Post Office had been to get a minimum of 500 amperes in the aerial; the station was working regularly with 500 to 550 amperes in the aerial, and still had a reserve of power.

Sir Guy Wrightson. Sir GUY WRIGHTSON thanked the Authors for their admirable Paper. He did not intend to touch on technical points, but there were two matters which, though they might not be regarded as engineering, counted as big factors in engineering. The first was what he might call the weather factor. On looking up the log he found that for about one-fifth of the time it had been impossible to go aloft on the masts because, although there might be no very strong wind on the ground, yet when aloft the wind was of considerable force. In addition, owing to the muddy state of the ground, it had often been impossible to move heavy weights about the site; that had hampered the work and had entailed duplicating the erecting plant in order to be in ample time for the completion of the erection. The second factor was what he might call the human factor. Light chain belts had been provided, and braces covered with leather, with hooks on the end, so that the men could attach themselves to the work if a sudden breeze arose. Sometimes they used them, but usually they did not. He often felt, however, that it gave much more confidence to a man to know that he was being looked after in every way possible to save him from accidents. In America the sky-scraper toll of life was measured by the number of storeys to an accident. In the case of Rugby nearly 10,000 feet had been erected with no toll of life at all. He regarded the Paper as the final laying of many ghosts, and although such points occupied only a few lines in the Paper, he could assure the members that they had often been the cause of sleepless nights for those responsible for the success of the erection.

Mr. Elwell. Mr. C. F. ELWELL regretted that he could not consider the masts to be a strictly British production from the point of view of design, as he had visited many stations where the masts were very similar to those under discussion. The Authors, in recommending the system of K bracing, stated that they thought it was better than single bracing; but he would ask whether they had considered cross bracing, which he thought was better than K bracing. Considering a section

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of the mast between a pair of stay-points as a uniformly-loaded beam, he thought that cross bracing would probably have been more effective. The Authors mentioned the circular section of post as being ideal. The circular section was rather tempting because of the reduction factor that could be employed. Several masts had been built in the past of circular shape, or with circular corner posts, but in his opinion the great difficulty lay in making adequate connections to the circular members. The corner posts in the mast shown in Figs. 2, Plate 7, consisted of two channels and a bent plate on the back. The ratio of greatest to least radius of gyration given was alluring, but his criticism was that it was practically a uniform section from the bottom to the top. That brought him to another point, namely, the question of the assumed wind load on the mast, which was 60 lbs. per square foot from bottom to top. Sir Guy Wrightson had pointed out that, when there was no wind on the ground, there was considerable wind high up the mast; and it was his own belief that there was a wind-gradient, and that if a certain wind was blowing on the ground it must be assumed that there was a stronger wind higher up the mast. He had allowed for 27 lbs. on the ground and about 60 lbs. per square foot on the top of a mast 820 feet high. In designing the Rugby masts it had been assumed that the stay-points would lie in a straight line; but how could that assumption be strictly correct if it depended on a constant wind-pressure over the entire length of the mast, which really did not exist in Nature. He thought it would have been better to substitute for the corner-post channels three or four angles back to back, reducing them, as the height increased, to three angles, then to compound stanchions, and finally reducing the angles. That would mean that the section offered to the wind would taper from the bottom to the top, and as the wind-pressure tapered from the top to the bottom, the product of the two was practically uniform. He believed that made for economy in design and also more precision in calculating the initial stresses, which must be done accurately if the mast was to lie in a straight line when the wind came upon it. He believed that the addition of a pivot was very necessary in a long cantilever of that kind; when the mast leaned over 1 per cent. in a gale there would be considerable cantilever action on the bottom of a fixed mast. But the kind of pivoting used at Rugby did not appeal to him, because the friction present would not do away entirely with the cantilever action. He used two convex surfaces, and the Daventry masts, which were pivoted on that system, gave very little friction. The method of making the stays was the same as was used in the German stations. The Authors did not mention the

Mr. Elwell.

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Mr. Elwell. number of wires, but he believed there were 150, of No. 10 gauge. Had the tests on the sockets shown in *Figs. 21* been made before the erection of the stays or after they had given trouble from individual wires slipping? It was rather difficult to hold 150 wires and make them all take their share of the stress. The Authors stated that a slight correction to the horizontal reactions was necessary, due to the wind load on the stays. His experience was that the wind on the stays, under some circumstances, could multiply the initial strain many times and add very serious loads to the masts. He congratulated the engineers on the solution of the question of mast-insulation by the use of a block of granite on a stanchion, but it was a question whether that was wholly effective, in view of the facts that the ends of the stays were insulated only by four 3-inch insulators in series, and that there were fifteen such units in parallel at each mast, forming a condenser at the earth end of the metallic system. He thought there was room for improvement there. Would the Authors give the weight of the masts?

Major Briggs. Major R. J. P. BRIGGS remarked that the subject of the strength of insulators, in which he was immediately interested, could be considered under three heads, namely, electrical, mechanical, and combined electrical and mechanical. It would appear that all the resources of British manufacture had not been taken into account in the selection of compression insulators. In compression tests made at Birmingham University in May, 1922, on a porcelain cylinder about 10 inches in diameter, with a $6\frac{3}{4}$-inch internal hole and a height of about $5\frac{3}{4}$ inches, a load of 90 tons had no effect on the porcelain. It was then placed in another machine and loaded to 312 tons; the insulator split into two parts, but did not fail by crushing. That particular insulator had not been ground quite true, and, but for that defect, he thought it would have withstood a considerably higher stress. It failed actually at 7·37 tons per square inch. Similar cylinders had withstood 8·75 tons per square inch, which was considerably higher than the figures shown for British materials on p. 226. The original arrangements for insulating the mast-stays at Rugby consisted of an "egg" insulator, with two grooves at right angles, into which steel ropes were interlinked. Mr. Shaughnessy, he believed, had had trouble with that type at Leafield some years ago. Egg insulators were particularly difficult to fire right through, owing to their thickness, and unless the ingredients in the porcelain body were vitrifiable, heat would develop in service, due to a sort of case-hardening effect in the initial drying, which locked up moisture in the cells of the porcelain. Eventually the insulator could be melted. In applying high-frequency tests with

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continuous waves, it was possible to bring the electrodes close together and produce very much higher electrical stress than if the electrodes were farther apart; so that it was conceivable that a bad insulator would give a comparatively good test if it were long enough. The shorter the insulator, the more essential it was that the porcelain should be absolutely vitrified right through. Some years ago he was privileged, by permission of the Post Office officials, to make tests at Northolt. Porcelain cylinders, about 8½ inches in diameter, 5 inches high, and approximately 1¼ inch thick, were soaked in water, and showed on continuous-wave test no rise of temperature after 8 hours. Some other solid insulators were tested at the same time, and a temperature-rise from 50° to 188°F. was obtained in 1 hour. It was not impossible to make a solid insulator that was thoroughly vitreous, but there was a definite limit to the thickness to which it could be made. The aerial insulators at Rugby had two interesting features. One was the extraordinary tensile strength of a porcelain tube, and the other was that the tubes were fitted with drip-rings, which ensured that rain-water would be immediately shed from the insulator instead of being allowed to run down on the under-side and so more or less short-circuit it. The actual weight of the complete insulated unit was 6½ cwt. The leading-in insulator weighed 146 lbs. and was 51 inches long. A novel arrangement for fixing was adopted by which porcelain rings were clamped by bolts of insulating material, so that there was no metal of any kind except the actual leading-up conductor. Corona-rings, if not used with very great care, were likely to lead to more trouble than might be anticipated, since there was a liability to focus the electrical stress on to the porcelain and produce a hot spot; he had seen an insulator from which the glaze had been completely removed at one spot from that cause. Major Briggs.

The AUTHORS, in reply, expressed their appreciation of Dr. Eccles's remarks and thanked him for his amplification of the Paper. They could assure Captain Eckersley that every part of the aerial-system had been tested before erection; and the spiders themselves had been tested for distortion under such conditions as would occur if one or more wires on one side only were broken. With regard to the use of a 6-inch sausage for the aerial leading-up trunk, the point was largely mechanical: the objection to using a solid conductor was that it would have to be comparable in size with a 3-inch or 4-inch copper tube, and the suspension of the necessary length at the required height was almost impossible. Captain Eckersley's question regarding the possibility of using an uninsulated mast appeared to have been based on a misconception. No part of the The Authors.

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The Authors. aerial was at a less distance than 10 feet from the masts. It had been actually proved, as the result of tests, that the ratio of the effective height with an insulated mast compared with a non-insulated mast was about 1·22 to 1. If Mr. Shaughnessy and Dr. Eccles failed to agree as to the variation in the frequency with temperature-rise of the tuning-fork, they hesitated to interpose, but the Authors thought it was something of the order that Mr. Shaughnessy had stated. There was no possibility of having stolen anyone's thunder in regard to the reading of the Paper. For obvious reasons it was quite impossible to cover in one Paper the whole of the problems that arose in the design, construction, and testing of such a station as Rugby. The Authors could heartily corroborate what Sir Guy Wrightson had said with regard to weather conditions. They did not think it would have been possible to choose a worse period for mast-erection, but the difficulties had been successfully overcome by the contractors. Mr. Elwell seemed to be under the impression that it was claimed that the masts were of purely British design. The Authors pointed out (p. 203) that the section had already been used in other wireless stations. The detailed design of the masts and the many novel features of the structures were, however, purely British. The Authors were inclined to agree that diagonal bracing, with the addition of horizontal members, would resist twisting equally as well as would **K** bracing: it would, however, probably offer a greater exposed area to wind. The Authors also agreed that the difficulty with circular posts was found in making the connections. There were many schools of thought as to what was a reasonable assumption to be made in determining the wind-pressure to be allowed for in such a structure as an aerial mast. They thought that no definite information was available as to wind-pressure at such heights and as to what the gradient of wind-pressure was likely to be. All recorded wind-pressures were at heights not much exceeding 300 or 400 feet, with the exception of one or two in isolated places. In the absence of more definite information, the only possible course was to make such allowance for pressure as had been made in the case under discussion, which was reasonably beyond the maximum wind-pressures that might be expected. Information was being obtained on the subject, and wind-pressures were being recorded up to heights of 300 feet. They hoped it would be possible at Rugby to ascertain actual wind-pressures at much greater heights. The shape of the section adopted for the vertical mast-posts was not ideal from the point of view of resistance to wind-pressure; it offered a considerable cup surface to the wind; but there were points in favour of a four-angle section with diminishing scantlings. Mr. Elwell was

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in error in stating that the section of the posts was practically uniform from bottom to top. Although the double-channel section was retained throughout the height of the mast, the weight per foot was very much reduced as the height increased, and that section had the great virtue of simplicity in manufacture and erection. The lower extremities consisted of 10-inch by 3½-inch channels, whilst the top section was composed of 5-inch by 2½-inch channels. With regard to the question of a graded wind-pressure, whilst the Authors concurred that wind-velocity near the ground was less than at high levels, they would point out that, so far as very tall masts were concerned, there was not much to be gained by assuming lower pressures within a belt of, say, 150 feet from the ground. The downward thrust in each section of the mast between stay-points, due to the stay-tension and its own mass, was added to the thrusts in the section immediately below. Thus, in the lowest section of the Rugby masts the compression due to the effects of the superstructure was about three times the maximum compression due to bending-moment on the section. Hence, little saving in section could be made by assuming lower wind-pressures on the lower section. The assumption of a straight line between stay-points, under maximum wind-pressure, was considered to approximate to actual conditions, even when uneven distribution of load obtained. The loading of the stays depended upon the heel of the masts as a whole, rather than upon the actual deflection of the material of the mast. Even under severe conditions of loading, deflection of the material would be of the order of inches, whereas the heel, even at the lowest stay-point, would be of the order of feet. Presumably the adoption of convex surfaces for the pivot of the Daventry masts was intended to permit of rolling action, since resistance due to pure friction was a product of the total load and the coefficient of friction. With the convex surfaces the concentration of pressure upon a small area was extreme. The consequences were twofold: firstly, lubrication was practically impossible; secondly, the material yielded, resulting in the formation of flats on, and even adhesion of, the metal surfaces. Thus, without in any degree reducing the vertical thrust on the pivot, the coefficient of friction was greatly increased. The total downward thrust on the Rugby masts was about 400 tons, or 100 tons per square foot of bearing surface. All surfaces of the pivot had been treated with anti-friction compound before erection. Further, owing to the low unit pressure, forced lubrication presented no difficulties. Unless special precautions were taken, two convex surfaces gave rise to an additional disadvantage, in that any side movement of the mast might result in the upper convex termination sliding down

The Authors.

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The Authors. the lower convex surface. Whilst it was correct that wind load on stays which hung between two fixed points might cause considerably increased tensions in the stays, the extent of the extra tension, in proportion to the initial tension, decreased as the diameter of the stays increased. In calculating the stay tensions, the effective mass per unit of length had been taken as the sum of the loading due to wind on the stays and the mass of the material. The latter, however, was much greater than the former. Moreover, by far the larger portion of the tension in the stays was due to wind load on the masts, which caused the upper ends of the stays to move and so reduce the sag. The tests on stays recorded in the Paper had been made concurrently with the construction and erection of the stays. It was not possible to test a 1,000-foot length of steel rope. The stays had been tested up to 125 tons, which was a little lower than their specified load, including the factor of safety. The total weight of mast and stays, with top gear and machinery, was about 200 tons. Major Briggs had suggested that the resources of British manufacture had not been taken into account in the selection of compression insulators. Such, however, was not the case. The figure given by Major Briggs for the load at which a compression insulator failed, namely 8·75 tons per square inch, was considerably less than the load withstood by the Danish insulator (Table IV, p. 226). It was known that British insulators could now show better results than those mentioned by Major Briggs. The use of "egg" insulators in stays at Rugby had never been contemplated, such insulators being entirely unsuitable for the severe conditions imposed. All tests upon insulators had been upon a pressure-gradient basis. The possibility, feared by Major Briggs, of smaller insulators being subjected to more severe tests than larger insulators, did not arise. The total insulation resistance was 12 megohms, including the stays.

The PRESIDENT drew attention to the almost unlimited power of the wind at considerable altitudes by recounting a personal experience he had undergone while mountaineering.[1]

[1] Minutes of Proceedings Inst. C.E., vol. ccxvi, p. 40.

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9 February, 1926.

Sir WILLIAM HENRY ELLIS, G.B.E., D.Eng.,
President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

GEORGE FREDERICK ATKINSON.
CECIL BURTON EDE.
LEONARD PERCIVAL HODGE.
HARMOOD VICTOR CARRUTHERS JOHNSTONE.
HAROLD LASHMAR PENFOLD, B.A. (*Cantab.*).
WILLIAM JOHN SHELLEY.
JOHN FREDERICK SIMMANCE.

And had admitted as

Students.

MEYER ADLER.
PHILIP JOHN ALLEY, B.E. (*New Zealand*).
PRAN NATH BATRA, B.A. (*Punjab*).
RICHARD THORNTON BETTS, B.Sc. (Eng.) (*Lond.*).
JACK HENTON BROCKBANK, B.Sc. (Eng.) (*Lond.*).
DOUGLAS HOWARD CASSIDY, B.A. (*Dublin*).
ARNOLD EDWIN CLARK.
CHARLES CLARK.
ROBERT WILLIAM CLERKE.
ALFRED D'AVRAY.
IRA ARTHUR DUBERY.
CHARLES RUSHTON ELLIOTT, B.Sc. (*Cape Town*).
HARRY GIBBON ETHERTON.
JOHN ALFRED FISHER.
EDWARD ALFRED FITZPATRICK, B.Sc. (Eng.) (*Lond.*).
HUGH LORAIN FOLKES.
GEORGE GALE.
JACOB GRUNBERG.
GEOFFREY HERBERT HARFORD.
D'ARCY ORMONDE HASKELL.
THOMAS HESKETH HIGSON, B.Sc. Tech. (*Manchester*).
ERNEST JOHN HILL, B.Sc. (*Birmingham*).
SIDNEY NICOL KELLY, B.Sc. (*Glasgow*).
MUHAMAD ABDUL KAYYUM KHAN, B.Sc. (*Allahabad*).
HUGH GORDON MCHAFFIE, B.Sc. (*Edin.*).
ERROL CYRIL EDWARD HOWE.
PHILIP GORDON HUDSON, B.Sc. (Eng.) (*Lond.*).
HENRY RANDON JOHNSON.
HARRY PERETZ KAUFMAN, B.Sc. (Eng.) (*Lond.*).
JAMES BURNS KIDSTON, B.Sc. (*Glas.*).
MALCOLM HAMISH LEES, B.Sc. (*Bristol*).
PETER MAULEVERER CRAWFORD LINDESAY.
JAMES WILLIAM LOWDON, B.Sc. (Eng.) (*Lond.*).
IAN MACALLAN.
NORMAN CECIL MACBEAN, B.Sc. (*Glas.*).
THOMAS ROBERT TYSON MIDDLETON, B.Sc. (Eng.) (*Lond.*).
JAG MOHAN, B.Sc. (*Edin.*).
CHRISTOPHER NEEDHAM, *Junr.*
ARCHIBALD GAVIN PARK.
HAROLD WILLIAM TALBOT PERKINS, B.Eng. (*Sheffield*).
BERNARD BOSWELL PFEIL.
VICTOR CHARLES FREDERICK PRICE, B.Sc. (Eng.) (*Lond.*).
WILLIAM LESLIE PRIESTLEY.

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Students—continued.

FREDERICK KELLY ROBERTS.
FRANK COLLINS SABIN.
WILLIAM JAMES SCOTT.
CLIFFORD PERCY SHELBOURNE.
PHILIP LAURENCE SHINGLER.
GEOFFREY SHRIMPTON.
COLIN HOPE SPENS.
MOHAMMAD HASSAN SYED, B.Sc. (*Punjab*).
RAM CHANDRA TANDAN, B.Sc. (*Manchester*).
WALTER ALFRED HENRY WATTS, B.Sc. (Eng.) (*Lond.*).
WILLIAM RISK WHITE.
ERIC WOOD.
LIONEL CLAYTON WOODHOUSE, B A. (*Cantab.*).

The Scrutineers reported that the following candidates had been duly elected as

Members.

JOHN WILLIAMS DOTY.
JOHN HENRY OSBORN EATON.

Associate Members.

GEORGE LLOYD ACKERS.
PERCY ROBERT ALDERMAN.
HARRY CLIFFORD ARMSTRONG.
GEORGE MUNRO ASHWELL.
EDWARD MONTAGUE AUSTEN.
LEONARD MILLER FRANK BARRETT, B.Sc. (Eng.) (*Lond.*).
CECIL WILLIAM EDWIN BISHOP, B.Sc. (Eng.) (*Lond.*).
PERCIVAL BENTON BRYDEN.
HARRY BURLEIGH, Stud. Inst. C.E.
CHARLES FREDERICK CHAMBERS.
MARIO TEIXEIRA DA COSTA.
JOHN ROGER DALLMEYER, B.Sc. (*Leeds*), Stud. Inst. C.E.
NORMAN KINGSLEY DICKIE, B.Sc. (*Glas.*).
ALEXANDER ROBERT ENTRICAN.
ARTHUR POST GRANT, B.E. (*New Zealand*).
MATTHEW EDMUND HABERSHON, M.Eng. (*Sheffield*), Stud. Inst. C.E.
ALFRED LAIRD HARVEY, *M.C.*, B.A. (*Cantab.*).
THOMAS CHARLES SKEFFINGTON HASLAM.
CHARLES HENRY MCLEOD HAWK.
BROOKE HODGSON.
WILLIAM ARTHUR INGHAM.
JOSEPH JENKINS, B.Sc. (*Queen's*).
CLEMENT MURRAY KERR, Stud. Inst. C.E.
CHARLES IDRYS KIDSON, B.E. (*New Zealand*).
GEORGE WILFRED BAXTER LOWSON.
RAE MACDONALD.
EDGAR RAVENSWOOD MCKILLOP.
DAVID ERSKINE SINCLAIR MASON.
THOMAS VYVYAN MORE, B.Sc. (*Birmingham*), Stud. Inst. C.E.
REGINALD WILLIAM MOUNTAIN, B.Sc. (Eng.) (*Lond.*).
ALFRED NICOLL MYLES, B.Sc. (Eng.) (*Lond.*).
MAURICE NACHSHEN, B.Sc. (Eng.) (*Lond.*).
GEORGE ALBERT PITT, B.Sc. (Eng.) (*Lond.*).
EDWARD WILSON RUSSELL.
ARUMUGAM SELVADURAI.
NINHAM SHAND, B.Sc. (*Cape Town*).
HOWARD MACOUN SHERRARD, (*Melb.*).
EDWIN MALCOLM SHIPMAN, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
ARTHUR RUSSELL SMEE, Stud. Inst. C.E.
REGINALD HENRY TRIVESS SMITH.
HUBERT GARIBALDI STEVENS.
STANLEY SWEENEY, Stud. Inst. C.E.
BERNARD GEORGE TWYCROSS, B.A (*Oxon.*).
NOEL EDMUND VINER VINER-BRADY, Stud. Inst. C.E.
FRANK WILLIAM FOSTER WAITT, B.C.E. (*Melb.*).

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Associate Members—continued.

JOHN DERWENT WALL, B.Sc. (*Manchester*).
RICHARD ELLIOTT PARK WHEATLEY, B.A., B.A.I. (*Dubl.*).
HARRY EDWARD WHITEHOUSE, *M.C.*, M.A. (*Cantab.*).
WILLIAM HARDY WHITEHOUSE, B.Sc. Tech. (*Manchester*).
DONALD ROY WILLIAMSON.
LEONARD WILSON, B.Eng. (*Liverpool*).
ALEXANDER WOODBURN, jun.
TREVOR MONCRIEF WRIGHT.
ROBERT MEREDYDD WYNNE-EDWARDS, *D.S.O.*, *M.C.*, B.A. (*Oxon.*).

The following Paper was presented for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

(*Paper No. 4557.*)

"Construction of the New Entrance to Tranmere Dock."

By PERCY WALTER BERTLIN, M. Inst. C.E.

THE Tranmere Shipyard, owned by Messrs. Cammell, Laird and Company, Ltd., is situated on the Birkenhead side of the river Mersey. It is provided with slipways, docks, and workshops suitable for the building and complete equipment of ships of the largest size. The wet dock is the largest fitting-out basin in the country, excluding those attached to the Government dockyards. This Paper describes the construction of the new entrance to this basin.

The outer or main dock has an area of about 15 acres and is in the form of a quadrilateral, one side of which is largely taken up by the entrance-works. The remaining three sides are available as quays, alongside which the ships lie for fitting-out or for repairs, the effective length of these quays being about 2,500 feet. Before the construction of the new entrance, the dock was provided with two entrances (Fig. 1, Plate 9), one having a clear width of opening of 91 feet 6 inches, with a depth of water of 33 feet 10 inches at H.W.O.S.T. and 26 feet 7 inches at H.W.O.N.T., and the other a width of opening of 30 feet, and a sill-level 12 feet 6 inches higher than that of the larger entrance.

Shortly after the outbreak of the war, a new slipway, 850 feet long, was completed. This placed the Company in a position to construct ships as long as, or longer than, any which had previously been laid down. In the early days of the war, however, the design of battleships and battle-cruisers was materially modified by the incorporation of "blisters" or bulges on each side of the hull, as a protection against torpedo attack, which materially increased the beam of the ship. One of these ships, with a beam of 104 feet, was laid down on the new slip, and to enable it to be brought into

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the wet basin to be fitted out, a wider and deeper entrance was necessary. The designs for this work were prepared by Messrs. Sir John Wolfe Barry, Lyster and Partners. The scheme provided for the elimination of the two existing entrances and the construction, in lieu of these, of one entrance with a clear opening of 140 feet and a depth, from high water to sill, of 38 feet 10 inches at H.W.O.S.T. and 31 feet 7 inches at H.W.O.N.T., closed by a rectangular steel caisson 145 feet long by 26 feet 9½ inches wide by 42 feet 9 inches deep. This is believed to be the largest dock-entrance in existence. The character of the work was such that it necessitated the carrying out of the construction in various stages and the completion of each stage before proceeding with the next. It is, therefore, proposed to deal with the construction stage by stage, and to divide the subject under the following six headings :—

(1) The construction of the main coffer-dam ;
(2) The construction of the work enclosed by the existing river-wall and the coffer-dam ;
(3) The removal of the length of river-wall in front of the site of the new entrance ;
(4) Launching and floating the new caisson ;
(5) The removal of the main dam ;
(6) The closing of the existing 90-foot entrance and the completion of the caisson-chamber.

It is intended to describe stage 1, namely, the construction of the coffer-dam, and also those stages of the work which appear to offer points of novelty, in some detail, and to give a more general description only of the remainder of the work.

The Coffer-dam.—The design of this structure presented considerable difficulties. The level of the impounded water at extraordinary spring tides was 24 feet above the old dock-sill, and the level of the bottom of the basin was — 20 feet, giving a depth of water of 44 feet. In view of the considerable head of water to be retained, and also of the fact that any failure of the dam would involve the grounding of, and probable serious damage to, a number of valuable war and other vessels, as well as probable loss of life, the matter was essentially one in which no risks could be taken.

The foundation of the whole of the works described in this Paper was red sandstone. A sheeted or puddled coffer-dam of any description presented many difficulties—not only in providing a satisfactory key to the rock bottom, but also in effectually strutting such a dam—due to the exigencies of the site. It was, therefore, decided that the only satisfactory solution of the problem was to construct a concrete-block dam; and this method commended

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itself even more strongly in view of the fact that a large proportion of the blocks could be used again in the construction of the permanent work at a later stage.

As the war was in progress, any idea of obtaining special plant in reasonable time was out of the question. The contractors therefore prepared designs for these temporary works to suit the cranes which they had available. The weight of the blocks was accordingly limited to 5 tons, which load the cranes were capable of lifting at a radius of 35 feet. The details of the interlocking and bonding of these blocks were prepared to suit the general section of the dam which was approved by the engineers, and the blocks were similar in general design to those which had previously been used at Gibraltar harbour and elsewhere (Figs. 2 and 3, Plate 9).

The next matter to be settled was the best method of making the dam watertight. The dam was sixteen courses high, and a total length of more than 2,000 linear yards of joints between the blocks was exposed to the water. It is clear that, with a maximum head of 44 feet of water, the problem presented difficulties, considering that all the work had to be done under water by divers. Further, the water was so discoloured that the divers were in total darkness directly their heads were 2 or 3 feet below the surface. Of course, the blocks could have been laid with cement-mortar joints and the problem dealt with in that manner; but, as the dam had to be removed again, cement joints were not desirable, and the progress would have been slower.

At Methil, a somewhat similar block dam had been constructed a short time before, and in that instance puddle joints had been used with some measure of success. It appeared, however, that previously-adopted methods might be improved upon, and that it was desirable to adopt some type of joint that would provide a watertight face, leaving the back joints open. This would avoid internal pressure and at the same time the result would be obtained with considerably less material to be handled by the divers, which was a point of much importance. Two trial blocks were made, with a semi-circular groove near the face of each, in which it was proposed to lay a roll of some material that would be plastic enough for the block to bed on to, and yet attain sufficient strength to resist the pressure of the water. Various mixtures and materials were tried, but none was entirely satisfactory. The experiment was then made of filling calico bags with dry cement and placing these bags or "sausages" in the grooves. It was found, however, that immediately the bags came in contact with the water they became too solid to be affected by the weight of the blocks, and the blocks failed to bed evenly

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on the cement sausage. This experiment, however, proved valuable as a stepping-stone. The blocks were next laid one upon the other, and an empty calico bag was placed in the groove. This bag was then filled with cement grout under pressure, and it was found that the bag had swollen tightly into the groove and that the desired result had been obtained.

The standard 5-ton block measured 7 feet 6 inches by 3 feet 8⅞ inches by 3 feet 1½ inch, but to meet the requirements of bond, etc., a number of other types of different sizes were necessary. The face blocks were all cast with a groove and splayed to receive the sausage, as shown in Figs. 3, Plate 9. The blocks were so designed that every course was interlocked into the course below it. Two wrought-iron suspension-eyes were cast in the blocks, sunk in recesses in the top surface. The blocks for the bottom course were provided with an interlocking recess on the top side only.

As no site was available in the shipyard for the manufacture of the blocks, a piece of land adjoining the northern boundary of Messrs. Cammell, Laird and Company's works was rented, which had a frontage to the river Mersey. The contractors there installed machine saws and mortising- and tenoning-machines, and at once commenced the manufacture of the necessary moulds. They provided at this block-yard two cranes, capable of lifting 5 tons at 35 feet radius, and also, on the river side, a 5-ton electric derrick crane, which was used for unloading the gravel from the coasting steamers and for loading the finished blocks into the barges for transport to the site of the dam. The block-moulds were made entirely of pitch-pine, strongly framed together, and great care was taken to ensure accuracy in the dimensions. Two sets of mould-bottoms were provided to each set of sides. The blocks were cast on their sides, so that the mould formed the top and bottom indent and only a level surface required screeding off. The concrete was mixed in a 1-cubic-yard electrically-driven concrete-mixer, and consisted of 7 parts of gravel to 1 part of cement. The gravel, which was obtained from Fleetwood and Piel, was delivered alongside the block-yard in coasting steamers.

After the moulds had been filled, the sides of the boxes were struck as soon as the concrete had set sufficiently (generally within 24 hours). The sides were then placed on the spare bottoms and refilled, the blocks first cast being left on the mould-bottoms a further 24 hours and then picked up by one of the cranes by means of special grips. They were finally deposited on their sides on a soft mattress of shavings. The cranes afterwards picked up the blocks again by means of hooks fastened on to the suspension eyes, turned

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them over into their correct vertical position, and deposited them on to the storage-ground to mature. Four strips of timber, about 2 inches wide by ¼ inch thick, were nailed on the upper surface of each block, while the concrete was still green, to form cushions. As far as possible, the blocks were matured for 3 or 4 weeks. It was necessary to cast special blocks, made to measure, for the closure of the dam. These were actually cast, picked up, transported, and set in the dam within 3 days.

The special grips, used for lifting the blocks while still in a green state from the box bottoms were employed in the following manner. As has been explained, the blocks were cast in the moulds on their sides, and as the blocks lay when cast, the lifting-eyes were on one side and therefore not in a convenient position for picking up. It was also undesirable to lift the green 5-ton blocks by means of the lifting-eyes, irrespective of the position of the latter. A grip was therefore designed to lift the blocks in such a manner as to take advantage of the compressive strength of the concrete and also to distribute the pressure over a comparatively large area of the block. The arrangement, which is shown in Figs. 4, Plate 9, consisted of two elm bars strengthened with steel plates, each of which was long enough to span the end of the block. The faces of the elm bars were grooved longitudinally to increase the friction when in contact with the surface of the concrete. By this means, directly the crane commenced to lift, the bars tightened on the block and held it firmly. The whole of the blocks cast were picked up with these grips and moved to the maturing-site without mishap.

While the first blocks were being made, the foundation for the dam was prepared. The site of the dam was dredged, and a deposit of about 5 feet of silt was removed as far as possible. The divers then washed the surface of the rock with fire-hoses supplied with water at high-pressure by a force-pump installed for the purpose. After thoroughly cleansing the rock, 56-lb. rails were laid longitudinally on each side of the dam to act as screeds, being packed upon small bags of concrete. These screeds were carefully levelled. The levelling-staff consisted of a 50-foot scaffold-pole with a heavy iron shoe, on the top of which an ordinary 16-foot staff was fastened. As soon as the screeds were levelled accurately, 3-to-1 concrete was lowered in skips having hinged doors at the bottom, carefully deposited between the screeds, and levelled by the divers, who used a weighted straight-edge to bring the concrete to an even surface.

The bottom course of blocks was laid upon a bed of 1-to-1 cement mortar on this concrete foundation, in order to prevent the dam

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from sliding. As an additional precaution against the passage of water under the bed-joint, a toe of bag concrete was laid in front of the bottom course of blocks.

The construction of the concrete dam was commenced at the two ends. Two 5-ton steam cranes installed, one at each end of the dam, picked up the blocks from the barges and lowered them to the divers, who set them in position. The dam was completed in 15-foot lengths. The bonding of the blocks in the wall was arranged so that a crane standing on top of the wall was able to build a complete length, and move forward on each length as constructed. A gang of two divers was engaged with each crane. One diver set the blocks and the other followed the block-layer with the insertion and grouting of the calico bags forming the sausage joints. The blocks were suspended from the cranes by double chains with a stretcher-bar, and the ends of the chains were secured to the suspension-eyes in the blocks by screw shackles to avoid accidents, which might have occurred had hooks been used. The average number of blocks set per week by the two gangs was 137, and the maximum number was 255 blocks, containing 742 cubic yards of concrete.

The calico bags for the cement-grout joints were made in about 17-foot lengths for the horizontal joints—sufficient to joint two stretcher blocks. One end of the bag was sewn up before it was inserted in the groove and grouted. The bags for the perpendicular joints were about 4 feet long. When these were inserted, care was exercised to keep the bottom and closed end of the bag well pressed on to the horizontal sausage with a certain amount of slack before the grout was injected. This ensured the vertical joint butting tightly on to the horizontal joint. The grout was mixed in a grouting-plant installed in a boat, and the grouting-pipe was lowered into the water. The diver tied the end of the calico bag on to the end of the grouting-pipe, and the grout was forced into the bag at a pressure of about 5 lbs. per square inch until the latter was completely filled.

The total time occupied in building the dam was 5½ months. It contained approximately 3,000 blocks, and the total quantity of concrete used in its construction was about 8,122 cubic yards. In addition to the plant already mentioned, a large steel pontoon was obtained, and upon this was installed a 5-ton crane, which was used in the first instance for placing the mass-concrete foundation and afterwards for setting the first course of the dam. The blocks of the first course, being laid in cement mortar, took much longer to fix than the other blocks laid without jointing material, and, there-

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fore the employment of two additional divers, working with the floating crane, materially expedited the work.

The Construction of the Work Enclosed by the Old River-wall and Coffer-dam.—While the construction of the dam was in progress, preparation was made for pumping out the water from behind the dam when completed. Before the water could be lowered between the river-wall and the new dam it was necessary to close the 30-foot entrance effectually against water-pressure from the river side. The 30-foot entrance-gates were, of course, designed to keep the water in the basin, but were obviously not capable, as fixed, of keeping the water out of the basin, which would become necessary as soon as the new coffer-dam functioned. The construction of a timber or steel dam across the entrance was considered, but a very cheap and effective method of avoiding this expense was adopted. On the sill of the entrance, and on the river side of the gates, a 12-inch by 12-inch timber sill was bolted and concreted in, having the same angle towards the river as the existing permanent sill had towards the dock. On the two side walls of the entrance on the river side were two rebates, and the ends of the temporary sill were placed to suit these rebates. On the closing of the main concrete dam, the gates of the 30-foot entrance were picked up by a 50-ton floating crane, turned round, and refixed in the new position prepared for them. Behind each heel-post a calico bag, 24 inches in circumference, was pushed down for the entire height of the heel-post, and into this grout was poured. The result was completely successful, and when pumping was commenced, it was found that this small dam and the cement heel-posts were absolutely water-tight.

Before lowering the water behind the coffer-dam, points were carefully established in a straight line on its crest, by which any deflection might be gauged. When the water was lowered, a deflection of about 1 inch took place at the top of the dam at the centre. In the Author's opinion a portion of this deflection was due to the compression of the timber slips, and it is doubtful whether they were advantageous from that point of view, although they were of considerable assistance to the block-setters in enabling them to insert their pinch bars under the blocks. When the water had been pumped out there was a certain amount of leakage, but the divers in two shifts caulked the worst places with oakum; and then a sprinkling of ashes and sawdust effectually sealed the dam. A 6-inch centrifugal pump working 25 per cent. of the time easily dealt with the whole of the water on the site, both that from the dam and the infiltration from the rock.

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The work of excavating and constructing the portion of the new entrance between the river-wall and the dam was now proceeded with (Figs. 5, Plate 9). The 5 feet depth of mud on top of the rock was first excavated by means of a grab, and after a few days it became sufficiently drained to enable hand excavation also to be carried out. The red sandstone had then to be excavated to a depth of about 16 feet. For the purpose of drilling this rock, and also for drilling the concrete, an Ingersoll compressor, having a capacity of 600 cubic feet of free air per minute, was installed, which was driven by a 110-HP. electric motor. The sandstone was drilled with jack-hammer drills and blasted with gelignite. This blasting loosened the rock sufficiently to enable it to be excavated by a Ruston-Proctor No. 10 crane navvy, which delivered the material into iron trays, each holding about 1½–2 solid cubic yards. These trays were lifted by long jib cranes, one of which took them from the navvy and placed them on a stage alongside the dam, and the other lifted them from this stage and deposited their contents in the barges lying alongside the dam.

On completion of the excavation, the concrete foundation for the sill was put in. When this was finished the granite sill-stones were set in position. These stones were of Cornish granite and of L shape, the horizontal side of the L forming the sliding face upon which the caisson was to rest. The vertical side formed the guides for the caisson, and the outer one also acted as the meeting-face with which the greenheart rubbing-pieces on the caisson made contact, thus forming the watertight seal. The vertical inner face of the caisson-sill for the width of the entrance, and both faces in the caisson-chamber, were dressed and polished at the quarry and fixed in position parallel to a tightly-strung wire. The horizontal faces and the vertical outer face of the outer caisson-sill for the width of the entrance were axed and polished after fixing. The results obtained in the two cases were, as far as could be ascertained, equally good. The dressing of the horizontal face was first done by the granite mason in the usual way with patent axes. The levels for the dressing of the horizontal sill were given with a 14-inch level, levels being read to $\frac{1}{80}$ inch on an ivory scale attached to the staff; and sights were invariably taken at even distances of 12 feet 6 inches from the instrument. The final polishing was done by drawing a steel plate about 1 inch thick, which had been previously planed to a true surface, backwards and forwards over the whole length of the sill, by means of a wire rope actuated by an electric winch. The plate was suitably loaded; steel shot was used for the first polish and carborundum and emery powder for the final polishing.

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The methods were quite successful, and both faces, when tested, were found to be extremely accurate. The largest local divergence from a true face, when tested with long steel straight-edges, was 0·006 inch, which speaks well for the care and accuracy of both the engineers and the granite masons.

While the second stage of the work was in progress, such other work as could be proceeded with was carried out, including the cutting-out of the dovetails for keying the old concrete to the new work (Figs. 1 and 5, Plate 9), the construction of the new roundheads, and the preparations for the removal of the outer river-wall. The cutting of the dovetails, although a matter of detail, may well be described, as it was a small operation that presented some difficulty in carrying it out rapidly and economically. The first method tried was that of boring a number of 4-inch holes for the full height of the wall on the plan outline of the dovetail, together with a hole in the centre of the piece to be removed. It was found that the holes, when drilled with a wagon drill, would be liable to diverge, and that truly vertical holes would not result. A Calyx core-drilling plant was, therefore, installed, and 4-inch holes were drilled with it. This plan, however, proved a failure. Calyx drilling was very slow, owing to small round pebbles in the concrete becoming detached from the side of the drill-hole and jamming the core bit, and, moreover, the holes did not have the effect of forming a weak line of cleavage. In short, it was found that the only quick and reasonably economical method was to drill a number of small holes with jack-hammer drills and to blast out the concrete piecemeal. When blasting, it was found almost impossible to preserve the acute angle of the dovetail. The engineers, realizing the difficulty, subsequently substituted the system of boring a number of holes about 24 inches deep into the old wall and cementing in 1 inch diameter steel bars, which projected into the new concrete an equal distance. These ties were generally spaced about 3 feet apart horizontally and vertically.

Removal of a Portion of the River-wall.—The blasting and removal of the existing river-wall constituted the third stage of the work. In order that there should be as little delay as possible in carrying out this work it was arranged that the whole of the necessary drilling should be completed in readiness. The river-wall was cellular in construction (Figs. 5 and 6, Plate 9) and consisted of two parallel walls with transverse tie walls 5 feet in width and 20 feet apart between centres. The cellular spaces had been filled with sandstone debris from rock level to the top of the wall. The transverse walls had been reinforced by three tiers of old bull-headed railway rails which were bent upwards into the front wall at one end, and into the

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back wall at the other end. These rails were a source of trouble, as in several instances drill-holes were bored to some depth only to find further progress impossible, owing to a rail having been encountered in an unexpected position. The rails also subsequently reduced the shattering effect of the explosives on the concrete of the transverse walls. Two rows of vertical holes, about 6 feet 6 inches apart longitudinally, were drilled through the entire depth of the wall. The distance apart of the rows of holes was about 2 feet 6 inches, and, as the holes in the two rows were staggered, the diagonal distance from hole to hole was about 4 feet. The holes were 47 feet deep. The plant used for this work was an Ingersoll wagon drill, which travelled on wheels and was provided with a turn-table base, reheater, and compressed-air hoist. The drill had a stroke of 8 inches and a bore of 5 inches, and worked at a pressure of 75 lbs. per square inch, delivering 250 blows per minute. The drills were solid hexagonal steel 2 inches in diameter, provided with cross bits. The size of the holes at the top was $4\frac{3}{4}$ inches in diameter and at the bottom 3 inches. For drilling a hole 47 feet deep a set of seven drills was used; the lengths of the drills were multiples of 7 feet 6 inches, the starting drill being 7 feet 6 inches long, and the last drill 52 feet 6 inches long; the bit was $\frac{1}{4}$ inch smaller in diameter at each change of drill. The washing-out of the hole was done by keeping the nozzle of a $\frac{3}{8}$-inch diameter water-pipe just above the top of the bit. Water was supplied at high pressure by a direct-acting Cameron pump carried on the drill wagon and driven by compressed air. The rate of drilling of these holes varied very much. In the drilling of a $4\frac{3}{4}$-inch hole at the commencement of a bore the usual rate was about 7 feet per hour, but the progress became slower as the depth increased, partly owing to time lost in changing drills, water-pipes, etc., and to the actual slower pace of drilling due to great difficulty in washing out the sludge, and also possibly due to less rigidity in the long drill steels. The average speed of drilling was about 40 to 45 linear feet per diem. When the drilling of each hole was finished it was filled with sand to avoid any possibility of stones or other obstructions blocking the hole before the explosives were inserted.

A timber dam was built across the end of the caisson-chamber (Figs. 5, Plate 9). This dam consisted of a bottom panel fitting the curve of the invert 29 feet by 6 feet, four rectangular panels of pitch-pine each 6 feet 9 inches wide by 21 feet 6 inches high, and four panels 6 feet 9 inches by 24 feet 3 inches high by 5 inches thick, supported by pitch-pine walings and struts.

On completion of the work described the whole of the plant

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within the area enclosed by the river-wall and dam was removed, the water from the river was admitted, and the two small entrance-gates were lifted out by means of a floating crane.

The top portion of the river-wall was next removed, down to a level about 2 feet above H.W.O.N.T., and loaded into barges, as it was considered better to dispose of the material in that manner than to allow it to fall into the river-bed to be subsequently dredged. The filling in the pockets between the front and back walls was also excavated down to rock-level, and the junction between the portion of the outer wall which was to be removed and the portion which was to remain was cut through and trimmed to the curve of the new roundheads, to within about 2 feet of the water face. This operation not only saved a great deal of underwater trimming but also reduced to a minimum the connection between the permanent work and the work to be removed by blasting, and, therefore, minimized any tendency to disturb the permanent portion of the work. After the removal of the filling, one 3-inch diameter hole was drilled into the sandstone rock in the centre of each pocket between the retaining-walls to enable the rock to be blasted and thus to facilitate its removal.

When the upper portion had been removed, the sand with which the holes had been filled was washed out with a hydraulic jet, and the holes were charged with explosives in the following manner. A charge of 6 lbs. of 50 per cent. gelignite, made up in 2-inch diameter pills, was lowered into the bottom of the hole and provided with a No. 7 submarine electric detonator. On top of this charge a canvas bag, filled with sand, and of such diameter that it would nearly fill the hole, was lowered. Further charges of gelignite and sand tamping followed, until the entire hole was filled. Fig. 6, Plate 9, shows a typical example of the charges and stemming of a bore-hole.

The electric wires attached to the detonator were rubber-covered so as to provide adequate insulation if they were submerged under water. Generally, two holes were fired together, and, whenever possible, they were selected so that the explosion was towards a free side. All the charges in the holes were electrically connected in series, which ensured the simultaneous discharge of each explosive unit in the two drill-holes. Immediately before finally joining the wires of the various charges, each circuit was separately tested with a galvanometer to make sure that there was no electric fault. The whole of the blasting was completed in about 3 days. The effect of the blasting was, on the whole, quite satisfactory as regards the breaking-up of the concrete, but in subsequent dredging operations it was found that a number of the large granite quoin-, sill-, and rubber-stones

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which had been built in the small entrance had not been fractured. The dredging was done by one of the Mersey Docks and Harbour Board's powerful bucket dredgers.

The Caisson.—The construction and launching of the sliding caisson does not call for any detailed description. The caisson was designed by the engineers and built by Messrs. Cammell, Laird and Company on one of their slipways, being constructed and launched on its side. The design and ballasting arrangements were such that the caisson could be used as a ship caisson and floated in and out of the recess until such time as the caisson-chamber was completed. The centrifugal pumps used for adjusting the water-level in the ballasting-tank were driven electrically by means of a flexible cable from the shore. After launching, the caisson was towed into the wet dock, moored alongside one of the quays, and there uprighted into its correct vertical position. It was then taken into dry dock, where it was fitted with greenheart sills and rubbers. It was afterwards brought into the wet dock, where it remained until it could be placed in position. This was done as soon as the dredging operations were completed.

Removal of the Coffer-dam.—Once the caisson was in position, the removal of the concrete coffer-dam was proceeded with. The blocks were hooked by divers, lifted into barges, and stored on land adjoining the inner basin. No difficulty was experienced in this work, and the whole of the dam was removed within about 8 weeks after the caisson was placed in position, when the new entrance was taken into daily use.

Completion.—The closing of the 90-foot entrance was next proceeded with. This work comprised the building of two walls at the back and front of the old 90-foot entrance, the space between them forming a part of the caisson-recess (Figs. 5, Plate 9). These walls were built entirely of the blocks which had formed the coffer-dam, and the work was carried out in a similar manner, except that the blocks were set in 2-to-1 Portland-cement mortar. On the completion of the walls the space between them was pumped out, and the construction of the remainder of the caisson-recess, with the granite sills, etc., was proceeded with, together with the drilling and blasting of the two portions of the old entrance which projected into the dock.

To control the water-level in the basin, three 8-foot by 8-foot sluices, were inserted in the river-wall on the south side of the new caisson-chamber. The invert-level of these sluices is 3 feet below the level of the old dock sill. Each sluice closed by two greenheart paddles working in polished granite slides. One set of paddles is

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controlled by hand gearing and the other set is opened and closed by electric power. The whole of the work of constructing these paddles and openings was done in the dry behind timber limpets, which were secured to the inner and outer sides of the river-wall.

Mr. A. R. Ellison, M. Inst. C.E., acted as Resident Engineer. The contractors for the work were Messrs. Edmund Nuttall and Company, of Manchester, who were represented on the ground by the Author, and for a portion of the work by Mr. R. H. Clarke.

The Paper is accompanied by three sheets of tracings, from which Plate 9 has been prepared.

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Discussion.

The President. The PRESIDENT, in moving the vote of thanks to the Author, asked whether there was any particular reason why 3 feet $8\frac{7}{8}$ inches had been adopted as a dimension of the standard pattern block. He suggested that it was not possible to work to $\frac{1}{8}$ inch in making large blocks such as those described in the Paper; and the engineers laid themselves open to the criticism that they were pretending to an accuracy which could not be attained.

Mr. Savile. Mr. L. H. SAVILE thought that the Paper was of particular value at the present time, on account of the high cost of constructing docks and works of a similar nature. Dock-companies and harbour-authorities were rather forced to do what they could to make their existing accommodation suitable for the larger type of ship by making new entrances or enlarging existing entrances, rather than meet the expense of building new docks, as probably they would like to do. For that reason any works of the nature described should be dealt with in Papers giving full information, each work being a distinct problem. He had described[1] in 1904 a somewhat similar work at Barrow-in-Furness, which had certain features of special interest. The Author had dealt with the work largely from the contractors' point of view, and Mr. Savile thought it was rather a pity that the engineers' reasons for the methods adopted in carrying out the work had not been given. It was a little difficult to see why the particular angle of the new entrance had been adopted; it was different from that of the old entrance. He would be glad if the Author would state the level of low water, as it was important to know the range of tide[2] which had to be dealt with. With reference to detailed construction, horizontal faces of the L-shaped stones were stated to have been polished. He had had considerable experience with stones of that nature, and he did not really see the necessity of polishing them, because the pressure of the sliding caisson on the stone was not very great, and there would not be undue wear. He would have thought that an axed finish would have provided a sufficiently fine surface. It would be of interest if the Author would describe how the stones had been set so accurately; whether they had been actually bedded down in cement mortar to the line,

[1] Minutes of Proceedings Inst. C.E., vol. clviii, p. 106.

[2] This has been added by the Author on Figs. 5, Plate 9 (*see* p. 265).—SEC. INST. C.E.

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Mr. Savile.

or wedged up to the right place, or whether mechanical means had been used for the final fine adjustment; and, if they had not been actually bedded in cement mortar, whether they had been only grouted on to the bed as well as on the sides? No provision appeared to have been made for any inner and outer stops. Should it be necessary at some future time to do any repairs in connection with the entrance, it might be that inner and outer stops and a spare caisson would be a good way of dealing with the matter; or, even if the spare caisson were not available, the provision of stops would render the building of a dam a relatively simple matter. It was claimed that the entrance was the largest in the world, and that was true to some extent, because it was the widest; but, taking the area at high water, he believed the Commonwealth dock at Boston was probably larger, namely, 133 feet by 44 feet 9 inches. The width of the entrance at Tranmere, 140 feet, might be considered even now to be rather on the large side. At the present moment the greatest beams were found in battleships, and the width of entrance that the Admiralty were adopting now for any new docks was 135 feet. The bulged ship had caused the recent increase in width, and certain ships were still being bulged. Why had the sliding caisson been adopted? The old entrance had gates. Why, also, had only one bearing-face been made? The caisson had a lifting deck, the advantage of which was possibly open to discussion. It would be of value to know what clearance was allowed between the caisson and the guides. The caisson was very wide, namely, 27 feet. A somewhat similar caisson at Barrow, for a 100-foot entrance, was 39 feet deep and only 12 feet wide; it did not seem necessary to have a caisson of the width described, except for the purpose of carrying traffic. The blockwork of the dam was similar to a type which the Admiralty had contemplated using a short time ago to close an entrance on a rock bottom. However, another method, which obviated the necessity of a dam, had been adopted, and would, he hoped, prove successful. The sausage used to render the dam watertight appeared to be very ingenious. Another method of getting the same results, which had been adopted by Messrs. Topham, Jones and Railton at Cardiff, was to tie some tarred rope round near the edge of each block. After setting, no diver's work was required, which was an advantage; and he believed that method had proved quite successful. A block dam was very costly unless the blocks could be used afterwards in other work, as in the present case, and it would be interesting to know whether, if there had been no further use for blocks of that sort, the cost of such a dam would have been justified. No costs were mentioned in the Paper; and

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Mr. Savile. if the Author could give any information on that subject, and as to the nature of the contract, it would add to the value of the Paper.

Mr. Cruttwell Mr. G. E. W. CRUTTWELL remarked that, although his firm had been the consulting engineers for the work described in the Paper, he had been engaged at the Ministry of Munitions at that time and had had very little to do with the dock, which had been dealt with almost entirely by his late partner, Mr. A. G. Lyster, Past-President Inst. C.E. Dealing first with Mr. Savile's question as to the angle of the entrance, the Mersey ran roughly north and south, and the entrance was nearly at right-angles to the direction of the river. He took it that that formed the best angle for ships entering: they came up principally on slack tide and went in at right-angles better than at any other angle. Polishing of the stones had been always his firm's practice. Naturally, a polished stone would offer less frictional resistance in hauling a caisson backwards and forwards than would a fine-axed stone, and he much preferred to see the stone polished. Even with a polished surface, the calculated hauling effort—which he presumed was more or less correct—amounted to no less than 70 tons. Mr. Savile had also remarked that stops had not been provided for use in the event of the caisson being damaged. It would be a very expensive matter to provide a spare caisson, which would be likely to get out of repair before it was wanted—if it ever was wanted. If at any time the caisson were damaged, he presumed the dock would be put temporarily out of action. Each side of the entrance towards the dock was splayed at an angle of about 45 degrees, and it would be practicable to fix a temporary coffer-dam across there without any stops. There was a total clearance of 2¼ inches between the timbers of the caisson and the rubbing-faces of the granite. The width of the caisson, less the sill and stem timbers, was 25 feet, which was considered necessary to provide stability, because the caisson had to be floated in and out for more than a year; a narrower caisson would have been unstable during the process of floating in and out. As to whether it was better to adopt gates or caissons, it was quite obvious from the plan why gates would not have been suitable in the present case. The width from the river face to the inside face of the dock-walls was only about 100 feet altogether; the presence of a pair of 70-foot gates rotating into that dock would have very much reduced the space available for ships. With regard to the Paper itself, he congratulated the Author on his very interesting communication. The Author stated that the coffer-dam was essentially a thing in regard to which no risks could be taken; and Figs. 2, Plate 9, showed that no risks had been taken. The base of the dam was 30 feet wide,

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as compared with a height of 40 feet, which was ample—some might say more than ample; but taking into account the fact that it was undesirable to use any mortar in the joints, and that the dam had to depend entirely on the mere weight of the blocks, it had been considered wise to adopt that ratio. The resultant line of pressure fell well within the inner third—about 11 feet 4 inches from the outer toe—even with the water in the dock at H.W.E.S.T. The contractors were to be congratulated on the success of their method of inserting the sausages, and also on the special grips by means of which the blocks were lifted. He had been surprised to see that the blocks were actually picked up, transported, and set in the dam within 3 days. He could only say that, if those particular blocks were not damaged, the grips must have answered their purpose. The bottom course of blocks was stated to have been laid upon a bed of 1-to-1 cement mortar on the concrete foundation. How had that mortar been spread? It seemed rather a difficult task for the divers, and any information on the subject would be useful. The Author also stated that the caisson bore only on the outer face of the granite: it could not bear on the inner face as it had to be floated out until such time as the caisson-chamber was finished. In order to float out the caisson powerful pumps had to be provided. There were three Tangye 16-inch centrifugal pumps, each driven by a 50-B.HP. motor, and each set was capable of pumping 5,200 gallons a minute. They were used not only for floating out, but also to pump the water into the dock to make up for leakage. They were capable, in 5 hours of raising the level of the water in the dock to the extent of 1 foot, which was the most the dock ever ran out during a tide. A Tangye 6-inch vertical three-throw pump, capable of pumping 730 gallons per minute, was also installed, principally for use during the floating-out process; that was required for trimming the caisson after it had been floated out of the recess, as the action of the 16-inch pumps was much too violent for the finer adjustments necessary. A point of interest about the caisson was that ten pipes had been provided, through which water could be sluiced from the dock, in order to disperse the silt which accumulated on the river side of the caisson to the extent of about 2 feet a year. The pipes had proved efficient, he believed, in keeping the front of the caisson clear of the silt, thus rendering its becoming jammed a very remote possibility. Those pipes did not have the effect of sluicing the entrance except close up against the caisson; and it might have been better if the 8-foot sluices had been put in the caisson itself rather than in the dock-wall, by which means all trouble of silting in the entrance would have been avoided.

Mr. Cruttwell.

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Mr. Reed. Mr. H. A. REED observed that the dam had a rock foundation, which made the work rather simpler than it would have been if the dam had been founded on sand or gravel, in which case it would have been necessary to adopt an ordinary coffer-dam. The Author stated that the moulds had been very carefully made, and Mr. Reed agreed that that was very necessary. When in Gibraltar he had made more than 100,000 such blocks, and great care had been taken to make them square and exactly of the right size. Divers could place such blocks much more quickly than if they were roughly made; and although accuracy in block-making was more expensive, it saved a great deal in the end. Mr. Reed had not used sausages in the way described in the Paper: the type used for putting a limpet on a dock-wall for repairs was generally made of sail-cloth, about 3 inches in diameter, and filled with oakum; but that sausage would not be suitable in the case under discussion, though it was very useful in other circumstances. With regard to the polishing of granite, he had always adopted an axed face for caisson-stops or lock-gate work, but for sluice-work polished faces were used for granite, with greenheart sluice-doors. About 1913 he started to replace the 12-foot by 6-foot sluices on the Manchester Ship-Canal. The work had to be done while the locks were in constant use, and he decided that all the stones should be polished at the quarries. The granite contractors objected, as it was their practice to rough out the stones in the quarry and polish them in position; but in this instance it would have been impossible to polish them in position. There was one stone in the invert, one in each side wall, and three in the arch. The rubbing-faces were 9 inches wide. He had rebuilt a great many of those sluices and had been able to make the polished faces absolutely watertight. He was not quite clear whether both the inner and the outer vertical faces of the caisson-stop at Tranmere were polished. Did the caisson bear both ways, inwards and outwards? How had the Author been able to get such long horizontal stops and the two vertical stops true and in the same parallel and plane to one another? It seemed a very difficult problem. The easier course would appear to be the usual one of having 1 inch for roughing off, putting in the true lines by wires, and working the faces in position. The Author used neat cement grout for the sausages, but Mr. Reed preferred 1-to-1 cement mortar for grouting work. Had the work been examined to see whether the cement had shrunk? He had had difficulty in blasting walls, owing to vibration being transmitted to near-by walls and ships. Had any damage occurred to work in the vicinity owing to the blasting? The Author stated that the moulds had been removed

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from the sides of the blocks after 24 hours, and that the blocks had been lifted after 2 days with a special apparatus. Had the block-making been done by the contractor for his own use in the dam, or was that procedure in accordance with a specification of the engineers? It seemed, from an engineering point of view, that to take the moulds off the sides of blocks at 1 day and lift them at 2 days was much too early. The question was specially important in this case, because the blocks had been used afterwards in the permanent work. With regard to making the blocks watertight in their permanent position, he failed to understand how a diver could make cement joints between the blocks under water. The joints would be only perhaps ¼ inch to ½ inch wide, and he did not think any mortar laid in that way would stay in position. Had the sausage been used as well as the cement joints? As to the Fleetwood gravel used to make the 7-to-1 concrete, his experience was that the quantity of sand contained in that gravel was very variable, and that the gravel varied a great deal in size. Had any special precautions been taken to get a regular aggregate? He would like the Author to say whether the 7 parts of gravel included sand, because if not it would mean a concrete mixture of about 1 : 3 : 7, which seemed rather weak for blockwork. Mr. Reed.

Mr. F. M. G. Du-Plat-Taylor observed that the entrance was remarkable for its extreme width. He believed the last two battleships launched, one of which had been built at Birkenhead, namely, H.M.S. "Nelson," had a beam of 106 feet, which included all bulges and other projections. Apparently it would not be possible under the Washington Treaty to build furthur ships of that size for a number of years, so that an entrance 140 feet wide seemed to be excessive, even for battleships. Comparing that entrance with others recently built, he found that the King George V dock entrance was 100 feet wide, a new dry dock at Havre, completed in 1925, 125 feet, and the Southampton floating dock 130 feet 8 inches. He believed the projected entrance to Tilbury dock was designed for 120 feet. On p. 242, the dam was described as a coffer-dam, but he had always understood that a coffer-dam was a box dam consisting of two sections of timber filled with clay. The dam described by the Author seemed to be simply a gravity dam, and should be described by its right name. Block dams were economical if the blocks could be used again. Many years ago in Liverpool a number of those dams were built in the tideway. The tides in the Mersey were very convenient for works, because they left the sills of most of the older dock-entrances dry at low water of spring tides. In those dams the joints were made of mortar and brown Mr. Du-Plat-Taylor.

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Mr. Du-Plat-Taylor.

paper, and it was quite easy to lift the blocks off and use them over and over again. How that would work under water he did not know. It might be possible, instead of using brown paper, to use wood-pulp or sheets of material that would not disintegrate whilst being laid under water. The blocks at Tranmere appeared to be very complicated when compared with the ordinary block used with brown paper and mortar joints.

The speaker then showed two lantern-slides of the original cofferdam, constructed under the late Mr. Anthony Lyster, Past President Inst. C.E., when he was Engineer-in-Chief of the Mersey Docks and Harbour Board, for the purpose of carrying out the work of the Tranmere Bay development.

It seemed to him that the pressure used in grouting the calico sausages was very low, considering the depth at which some of the sausages were inserted. Apparently there was a certain amount of leakage through the dam, because the Author mentioned that leakage was stopped by the usual means. A good many years ago he made use of a sausage, somewhat similar to that described, in the construction of two lock-entrances to the Brunswick dock at Liverpool. In the course of that work the apron of the existing Toxeth lock alongside had to be removed. A heading was being excavated in the old apron, and in the course of that excavation an old 6-inch construction-drain burst. The drain was in communication with the Toxeth dock, and in 2 or 3 days the whole of the works behind the dams were drowned to a depth of 44 or 45 feet. Eventually a canvas sausage, about 16 feet long and 6 inches in diameter, was pushed into the construction-drain by a diver, connected with grouting-pipes, and filled with grout under a pressure of 25 lbs. per square inch; and that was successful. It was simply held in place by the pressure and the friction between the canvas bag and the side of the drain. After it had been there for 3 or 4 days, the works were pumped out, and there was no further trouble.

Mr. Varley.

Mr. GILBERT VARLEY remarked that, as he had been responsible for the operation and maintenance of the entrance since its completion, there might be some points on which he could assist the discussion. The question of the width of the entrance of the Tranmere dock was rather a peculiar one. The flow of tides in the Mersey opposite the entrance was such that 1 hour before high water there was actually an ebb opposite the entrance, the result being that, as the entrance could only be opened at the most 1½ hour before high water, it was generally found that ships coming into the entrance had a fair amount of tide tending to set them towards

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Mr. Varley.

the north; hence the entrance was inclined as much as possible towards the south. At the same time the wide entrance had been found to be of great advantage in getting ships in. Mr. Cruttwell had mentioned that three pumps had been put into the caisson to make up for any leakage from the basin. No leakage, however, had been detected, and the pumps had never been used for impounding. The caisson was used in the first instance as a ship caisson, and the whole entrance was so designed that it was capable of being used in that way. There was one difficulty. In designing the caisson, provision had not been made for any material amount of buoyancy towards the two ends, and in pumping out the caisson there was a tendency for it to come up one way or the other; and even with the small pump it was not possible to control the caisson sufficiently closely to bring it up on an even keel. Therefore two extra bouyancy-chambers had been put in, one at each end of the caisson, which were always kept dry, and thus the buoyancy was maintained, which kept the caisson level. The method of operating the caisson was to control the preponderance of the caisson, which ranged between 20 tons for neap tides and a maximum of 140 tons for extreme spring tides. That was done by the pumps previously mentioned, and no difficulty whatever was found in varying to limits of 1 inch the level in the tank during the tide. It was not an uncommon thing to vary the quantity of water between opening and pumping to the extent of 3 or 4 inches. Polishing the sill stones had shown itself to be an advantage, in that up to the present day there had been no sign whatever of any wear of the greenheart timbers; the original markings made for checking this wear were still lineable. Another advantage had been the reduction in the power required to haul the caisson. Including all losses in the motors and gear, the coefficient of friction was 0·5, and that included any wave-action in the caisson-chamber itself. The speed of opening the entrance at the maximum preponderance of 140 tons was about 20 feet per minute. The plant was designed to deal with twice that preponderance at 15 feet per minute, but with the lighter preponderance of 20 tons a haulage speed of 30 feet per minute was obtained. Mr. Cruttwell had raised an instructive point which was becoming very important at Birkenhead, namely, the question of silting opposite the entrance. About 2 feet of mud was made in 12 months. The orifices in the caisson had proved ineffective in sweeping the mud away, and as time went on the silting was worse. In front of the three 8-foot sluices there was a hole 13 feet deep over an area 50 feet in diameter, and it looked as if it would be necessary in the future to fit the sluices into the

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Mr. Varley. caisson itself, and thereby get the advantage of their scouring effect opposite to the entrance.

Mr. Deane. Mr. H. J. DEANE asked what was meant by the "pressure of about 5 lbs. per square inch" employed for grouting. Was it the preponderance over the water-pressure at the bottom course, or the actual pressure as recorded by the gauge on the grouting-machine? What had been the condition of the sausages when the blocks were removed? Had they shown signs of having completely filled the grooves or not? In his own experience—which was perhaps fairly extensive—of grouting under pressure he had found it quite useless to use low pressures if a perfectly tight joint was required. On the tube railways, for instance, the grouting was done under pressures up to about 60 lbs. per square inch. The depth of water under the particular dam under discussion was 44 feet, and therefore there would be a pressure of about 20 lbs. per square inch at the bottom; consequently it would appear that at least 60 lbs. per square inch would be required at the gauge to make a satisfactory job. On the question of the material used for the grout, he thought it was general experience that, when cement alone was used, it was liable to shrink, and the sausages would not completely fill the spaces. On tube-railway and similar work it was common practice to use hydraulic lime, which, he considered, in view of its expansion on setting, would prove very valuable, with high pressure, as a means of securing a perfectly tight joint in the dam. If that practice had been adopted, he believed the slight difficulty experienced at Tranmere would not have occurred.

Mr. Howorth. Mr. BEN HOWORTH remarked that the Paper was particularly interesting to him, as he had been a pupil of the engineers of the original scheme, and had been employed on it from its beginning until its completion in 1907. When the scheme was first brought forward there was much discussion as to what the width of the entrance should be, and Mr. J. T. Wood, M. Inst. C.E., advocated originally two entrances, one of 100 feet and another of 80 feet; but, as Mr. Wood had pointed out in the discussion which took place at the Institution on the Paper[1] read on the original work, it was a commercial undertaking, money was a serious consideration, and his scheme for two entrances was ruled out. Then he suggested one entrance of 100 feet; that also was ruled out, and eventually the two entrances were built, namely, 91 feet 6 inches and 30 feet. Mr. Wood's forecast of 100 feet as a suitable entrance-width was

[1] S. H. Ellis. "The Tranmere Bay Development Works." Minutes of Proceedings Inst. C.E., vol. clxxi (1907), p. 127.

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particularly accurate, as even now there were only eight steamers with more than 90 feet beam—two just touched 100 feet. A width of 90 feet had been advocated as early as 1862 by Mr. James Abernethy, Past-President, for docks on this site. As a matter of fact, the width of 91 feet 6 inches was originally started as a 90-foot entrance, and the walls had been carried up 20 feet when it was decided to cut each side back 9 inches and to dress the granite right down to sill-level, thus making the 91-foot 6-inch entrance. At the International Navigation Congress in 1923 a number of prophecies as to future dimensions of vessels were made, which were worked out from tabulated results; among them a beam of 160 feet was mentioned as possible in the middle of the present century, say 1942, for abnormal Atlantic steamers. It was thought that normal Atlantic steamers would reach about 110 feet beam during that period. He therefore considered that 140 feet was certainly a very generous estimate. Mr. Howorth.

Mr. JAMES MITCHELL considered that the dam was well adapted to the circumstances, and showed a clear appreciation of the distinction between the two primary requisites of a dam—watertightness and stability. The thickness of the watertight portion was only about 3 inches, and formed a very striking contrast with a clay dam having a thickness of perhaps twice that number of feet. Like the President, he was rather struck by the $\frac{7}{8}$ inch in the dimensions of the 5-ton blocks. The contractors were to be congratulated on the setting properties of their concrete when they were able to lift 5-ton blocks after 48 hours, even with the excellent type of lifter adopted for the purpose. They were perhaps also fortunate in that their experiments with pre-formed jointing-material were a failure. Even supposing the jointing-material to be uniform in section and in compressibility, it would be impossible to set the blocks so that the grooves would be uniform in section, and there would in consequence be a danger of not getting the blocks set evenly. The Author stated that the deflection which took place after pumping out was probably due to the timber slips that were nailed to the blocks for the purpose of distributing pressures in the maturing-yard. In addition, however, to helping the divers to set the blocks, the strips would be valuable for equalizing pressures in the dam, especially having regard to the fact that the blocks were later to be used in the permanent work. Those pressures would be very considerable at the base of the wall. He was always rather surprised at the use of dry and semi-dry cementing materials under water. As far as his experience went, it was even more important to use fully-saturated materials under water than above, because above water Mr. Mitchell.

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Mr. Mitchell. some compensation could be made by ramming. Like Mr. Savile, he was struck by the difficulty of setting the previously-polished stones with an accuracy of 0·006 inch as the maximum divergence. Although it had produced a very good result, he did not think the method adopted for grinding and polishing the stones in place was quite the best. Probably a better method would have been that adopted in ordinary machine-shop practice, where a surface-plate or template smeared with paint was used to define the high places, which were then attacked piecemeal by filing, scraping, or grinding. By that method any required degree of accuracy could be obtained. Similarly, in truing up the greenheart meeting-faces of dock-gates, the surfaces, after being planed as accurately as possible, were spotted by means of templates and straight-edges, and the high portions were removed by means of steel scrapers. He thought the method adopted in grinding tessellated marble pavements would be well adapted to the polishing of granite, the surface being spotted by templates and straight-edges and the high portions ground down by rotating carborundum-wheels driven by flexible shafts from suitable motors. The cutting of the dovetails for connecting the old wall to the new seemed to have been a troublesome job, and he had no doubt there would be considerable difference of opinion as to the advantages of longitudinal as compared with breast drilling, and as to whether blasting was better than bursting out by means of hydraulic jacks. The method finally adopted—that of connecting the old work to the new by means of steel rods—was, he thought, better and more economical than dovetailing. On looking recently at some work done about 15 years ago in refacing a portion of a concrete breakwater by that method, he had found it quite as good as when originally carried out. Another interesting part of the work described was the use of the small dock-gates as a dam, by setting them against a temporary sill and then grouting the heel-post opening by means of neat-cement grout. His experience had been that in grouting very good work could be carried out by means of neat cement—much better, he thought, than with sand and cement. A great deal, however, depended on the consistency of the grout, which was frequently made much too thin. It ought invariably to be as stiff as could be used for the particular kind of work required, and to be very thoroughly mixed.

The Author. The AUTHOR, in reply, expressed his appreciation of the kind way in which the Paper had been received. The dimension 3 feet $8\frac{7}{8}$ inches had been adopted for the width of the blocks because for the purpose of bond it was necessary that the width of two blocks laid as headers, plus $\frac{1}{4}$ inch for the joint, should equal the length of a block laid

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The Author.

as a stretcher, which was 7 feet 6 inches. The blocks had, in fact been cast with great accuracy, and certainly they were true to size within $\frac{1}{16}$-inch. He would be glad to add the range of tide in Figs. 5, Plate 9, in accordance with Mr. Savile's suggestion. As stated on p. 248, the L stones had been accurately fixed in position by being set on iron packings and wedges, the final adjustments to line and level being made with extreme accuracy by means of pinch bars and slightly tapered iron wedges. Both the vertical joints and the bed joints had been grouted with cement. In his opinion the use of a block dam on the work described would have been justified even if it had not been possible to incorporate the blocks in the permanent work. If the position of the dam were considered, it would be seen how difficult it would have been to construct any other type of dam which could be safely supported in such a manner as to leave the site free for the construction of the permanent works, and at the same time encroach so little on the area of the dock. Moreover, it had been impressed on both the engineers and the contractors that the dam should be so designed and constructed as to eliminate every possible risk. As to the cost, the work had been carried out during war time, and there had been such wide fluctuations in both the rates of wages and the costs of materials during its progress, that any unit costs would be absolutely misleading. The contractors had carried out the work on a basis of cost plus a fixed remuneration. The 1-to-1 cement mortar upon which the bottom course of blocks was laid had been lowered to the divers in small skips with bottom doors. When the skips rested on the concrete, the doors were opened, and the mortar was allowed to fall gently on to the concrete. Each skip contained about $\frac{1}{2}$ cubic foot of mortar, and several of these small heaps were deposited on the site which was to be occupied by a block. The heaps were gently pushed down, forming a fairly even bed of mortar upon which the block was lowered. Some doubt had been expressed by several speakers as to the possibility of putting in such a joint effectively under water. It had been found, however, when the dam was taken out, that the cement joint under the bottom course was quite satisfactory. As Mr. Reed had remarked, the rock foundation was certainly an advantage, in fact one might go as far as to say a necessity, in the case of a concrete block dam. Had the site been sand or gravel, possibly the only way of overcoming the difficulty would have been to encroach further upon the space of the dock in order to give sufficient room clear of the permanent work to enable rakers to be got in at suitable angles to support a pile dam. The divers engaged upon the work had been highly-skilled men who had carried

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The Author. out very similar block-laying work in various parts of the world, and they had acknowledged that the precision with which this wall had been built under very difficult conditions was largely due to the extreme accuracy of the blocks, which they stated were the best blocks they had ever laid. Both inner and outer faces of the caisson-stop at Tranmere were polished. The caisson did not bear both ways, but only outwards, although both inner and outer faces functioned as guides if the caisson was not travelling in a straight line and happened to be affected by either wind or current. With regard to the accuracy of the long horizontal stops and the two vertical stops, as stated on p. 248, the vertical outer or pressure face had been polished in situ; the inner vertical face of the entrance and the vertical faces in the caisson-chamber had been polished at the quarry before fixing. All the polished faces had been accurately set to a wire. The two horizontal faces as received from the quarry had been axed and set about $\frac{1}{8}$ inch high to allow for dressing off, fine axing, and polishing after the stones were fixed. The method adopted, in the Author's opinion, had not only been very successful, but had also saved time and proved very much cheaper than the ordinary hand polishing.

With regard to the grout used for the sausages, experiments had been made with neat cement and 1-to-1 mortar; the neat cement was found to give the best results, there being not the slightest trace of any shrinkage. No difficulty whatever had occurred owing to vibration caused by blasting being transmitted to nearby walls or ships, and no damage had been caused to work in the vicinity. The block-making had been carried out to the satisfaction of the engineers, but reasonable discretion had been allowed to the contractors. The work being one of national importance, both the engineers and the contractors, in order to avoid delay, had used every expedient that it was considered could be adopted without detriment to the permanent work. The best proof of the satisfactory character of the blocks was the fact that, after being set in their temporary position, and then taken out and stored, they had proved, when relaid and set in their permanent position, to be excellent and sound in every respect. Pieces cut from the blocks had shown that the concrete was of first-rate quality; not a single block had been found which was not thoroughly sound. No sausage had been used in the bed joint, but, as already stated, that joint had been found quite satisfactory. The concrete consisted of 7 parts of aggregate, consisting of sand and gravel, to 1 part of cement. The width of the entrance might appear to be extreme when compared with other modern and projected entrances, but

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The Author.

as had been pointed out by Mr. Varley, the flow of the tides made docking and undocking ships at this point of the river so difficult, that the width decided upon seemed to have been justified by the ease with which ships could now be manipulated; the round head on the north side of the entrance had also been found to be very advantageous. He agreed that the dam would have been better described as a concrete-block dam. He was strongly of the opinion that the form of joint adopted for making the dam watertight had proved to be cheaper and more satisfactory than any system of mortar joints laid under water would have been. Moreover, laying blocks on a mortar joint proved very much slower, required very much greater care, and, as pointed out in the Paper, would have involved the handling of a considerably larger quantity of material by the divers than the system adopted. The pressure used to grout the calico sausages had been found in practice to be sufficient: not a single bag had been found which had not been tightly swollen into the groove in such a manner as to perform its work efficiently. Mr. Deane had perhaps overlooked the fact that without any air-pressure in the grouting-pan there was very nearly sufficient pressure to fill the bag efficiently, due to the difference between the specific gravity of the cement and that of the water in the vertical grouting-pipe. The 5 lbs. per square inch was the pressure recorded by the gauge on the grouting-machine. That such a low pressure had been found to be quite adequate in this case was probably due to the fact that it was not a question of forcing grout into fine crevices, but simply one of filling a comparatively large tube to which the grout had very free access. The entry of the cement immediately forced any air or water that might be in the bag through the pores of the material, and a solid core of cement was at once formed. The Author considered that the use of lime would not have been satisfactory, as there would have been the possibility of the bags bursting, due to expansion, and of subsequent trouble being caused thereby. Mr. Mitchell's remarks concerning the use of dry and semi-dry cementing materials under water were not quite clear to him, as in no instance had such materials been used in the manner indicated. With regard to the suggested polishing of the horizontal faces by piecemeal methods of grinding away, the wonderfully accurate results obtained by means of the steel plate described in the Paper appeared to have quite justified its use, especially in view of the fact that the work had been carried out by this method very much more cheaply and expeditiously. When the polishing had been completed by the methods described in the Paper, it was

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The Author. checked with templates and straight-edges in the manner described by Mr. Mitchell, and no subsequent local grinding down had been found necessary. For cutting the dovetails in the old wall hydraulic jacks had been experimented with, but not successfully. He was obliged to Mr. Varley for the interesting information given as to the working of the caisson since its construction.

Correspondence.

Mr. Halcrow. Mr. W. T. HALCROW considered that blockwork coffer-dams had obvious advantages where the blocks could be utilized later in the permanent work. The chief difficulty to be overcome in their construction was the making of a watertight face. The method described by the Author, which appeared to have proved very satisfactory, was ingenious. In the case of a similar coffer-dam for a dock at Swansea, designed by the late Mr. C. S. Meik, M. Inst. C.E., some years ago, watertightness had been obtained by driving a row of steel sheet piling in front of the blockwork, and filling the space between the sheet piling and the blocks with puddle clay; that dam had proved entirely successful. It would be of interest if the Author could give figures as to the cost of making the dam watertight by the method adopted at Tranmere.

Mr. Wilson. Mr. MAURICE F.-G. WILSON congratulated the Author on an interesting Paper, clearly expressed, so that the various operations described could be easily followed. A Paper of this nature, which dealt with the alterations and reconstruction of existing work, executed in such a manner as to allow the general traffic to be carried on without interruption, was always of special interest. In the present case, everything seemed to have been very carefully thought out, and the whole of the work carried on without any hitch or special difficulty, although it had been considerably handicapped—as all works carried out at that period had been—in regard to the difficulty of obtaining special plant during the war. In one respect, however, the works at Tranmere dock were in a fortunate position, inasmuch as they had a good rock foundation throughout. Good foundations disposed of nine-tenths of the difficulties in carrying out works, especially docks and similar undertakings, which were so often placed in positions, such as estuaries and marsh land, where the underlying strata were soft and unsuitable, in which case the foundations gave

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very considerable trouble. The concrete-block coffer-dam was a very interesting structure, especially in regard to the method adopted for making it watertight; that was always a difficult matter, and the difficulty seemed to have been overcome satisfactorily and in an ingenious manner. Whether the specially shaped blocks, so formed as to key into one another in cross section, had been absolutely necessary, was perhaps doubtful. There would not have been much risk of plain rectangular blocks sliding, and they would have been easier to make and to set. At the same time, the additional security afforded by the keyed blocks perhaps compensated for the extra time and trouble. Another matter of some doubt was whether there was any advantage in polishing the caisson-stops. That must always be a considerable expense, and it was doubtful if such a face made a better watertight joint with the timber work of the caisson than did an ordinary fine-axed face. Very rarely could the stones be set so truly as to obviate any trimming of the face after they had been placed in position, in which case the polish was, of course, wasted. It would be interesting to know if in the present case the stones had had to be re-dressed to any extent after having been set. The drilling and blasting away of the old dock-wall seemed to have been carried out very successfully, in spite of the great depth to which the holes had been drilled—nearly 50 feet. It would have been interesting if the Author had given the total volume of concrete removed per pound of explosive. The subsequent use of the blocks forming the coffer-dam for building the new river-wall was certainly a good and economical arrangement. He noticed that these blocks, when built into the quay-wall, had been set in mortar. The advantage of this arrangement was doubtful. It was certainly costly, and it was extremely likely that portions of the mortar beds had been washed out. It would have been much more usual to set them in the dry, without any attempt at mortar beds. If it had been necessary to make the wall specially watertight, this could have been done in the same manner as when the blocks were used for the coffer-dam. The object of the small wood strips which were attached to the blocks was not quite clear. The whole weight of the blocks would come upon the wood strips, instead of being distributed over the general surface of the blocks as it would have been if the strips had not been used.

Mr. Wilson.

The AUTHOR, in reply, observed that the approximate cost in labour and materials of making the dam watertight with the cement sausages had been 4*s*. 6*d*. per square yard, or 22*s*. 6*d*. per linear foot of dam. At the time when this work was done the labour rate for a diver ranged from 2*s*. 6*d*. to 3*s*. per hour. In his opinion the

The Author.

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The Author. use of the interlocking type of concrete block adopted had been justified in practice, if only by the readiness with which divers could lay it accurately in the discoloured water. None of the stones which had been polished at the quarry and set to a wire had required any subsequent re-dressing. The volume of concrete in the old dock-wall removed by blasting was 1 cubic yard to ½ lb. of 50-per-cent. gelignite. With regard to setting the blocks in cement mortar, when built into the quay-wall the larger part of that wall had been constructed tidally. As its purpose was to retain the water in the dock at low water, the wall was subjected to a considerable head. There were practical difficulties in placing the cement sausage joints on the inner or dock face of the wall; and, as the sausage joint was used on the outer face only, it meant that, had the joints not been filled, the water would have entered them and subjected the wall to internal pressure. It would therefore appear that the adoption of the system of filling the joints solidly with cement was justified. The timber slips had been adopted with the object of forming a compressible cushion between the blocks, but the Author agreed with Mr. Wilson that their advantage was doubtful, except in so far as they had been of assistance to divers in setting the blocks, as indicated in the Paper.

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OBITUARY.

Captain MATTHEW HENRY PHINEAS RIALL SANKEY, C.B., C.B.E., R.E. (retired), the son of General W. Sankey, C.B., was born at Nenagh, Co. Tipperary, on the 9th November, 1853. He was educated in Switzerland and at Mr. Rippin's school at Woolwich, and in 1871 entered the Royal Military Academy, Woolwich, where he passed out at the head of the list and won the Pollock Medal and the Sword of Honour. He received his commission in 1873, and in 1874 proceeded to the School of Military Engineering, Chatham, where he showed marked aptitude for mathematics. During his course at the School, he was appointed to assist in the conduct of the trials of continuous railway-brakes, carried out by the Royal Commission on Railway Accidents at Thurgarton, and he was subsequently engaged in the office of the late Mr. Edward Woods, Past-President, on calculations in connection with the trials.

On leaving the School of Military Engineering, he was first employed at the War Office on architectural design, and was sent later to Manchester, in charge of a drawing-office employing sixteen draughtsmen. Thence he proceeded to Gibraltar to superintend the construction of works and erection of barracks in connection with the Military Telegraphs and Signal Station. While at Gibraltar he was appointed, in 1879, as an instructor at the new Royal Military College, Kingston, Canada. He did valuable work in initiating additional courses of instruction, and during this period he published a text-book on electrical measurements.

In 1882 he was recalled and sent to the Ordnance Department at Southampton, where he at once encountered a technical problem in connection with the electrical deposition of copper in the process of duplicating the plates from which maps were printed. Attempts had been made to substitute a dynamo for the costly primary cells which supplied the current for the process; but these had proved disappointing. Captain Sankey was able to surmount the difficulties, and he was also responsible for the introduction of power presses, which, by obviating the necessity for damping the paper, enabled the maps to be printed quite true to scale. In the course of his work at Southampton he required a high-speed steam-engine, and his search for a suitable design led to his acquaintance with the late Mr. P. W. Willans, M. Inst. C.E., which subsequently had so marked an influence on his life. In 1889, at Mr. Willans's suggestion, he resigned his commission and joined the board of Messrs. Willans and Robinson. He was placed in charge of the erection and running of all the engines produced by the firm, and he took a leading part in the important thermodynamical investigations which formed the subject of the second of Mr. Willans's

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two communications to The Institution on Steam-Engine Trials. As a result of occasional, and at that time mysterious, crankshaft failures in the engines, he was led to the investigation of crankshaft stresses. He published later,[1] in collaboration with Dr. C. Chree, F.R.S., and Mr. W. E. W. Millington, M. Inst. C.E., the results of these researches, for which he was awarded a George Stephenson Medal.

On the death of Mr. Willans in 1892, Captain Sankey became responsible for the design of the engines and steam-turbines which the company were then producing. In conjunction with Mr. Willans, he designed comprehensive testing equipment, which was practically pioneer work at that time, and he was responsible for improvements in apparatus and in methods of calculation which have since been widely adopted. The old Thames Ditton works of the company being inadequate, the selection of a site for new works became necessary, and Captain Sankey was mainly responsible for the layout of the Victoria Works at Rugby which were completed in 1898. In 1904 he resigned his appointment to take up consulting work in steam and internal-combustion engineering. In 1909 he joined the boards of, and was appointed consulting engineer to, Marconi's Wireless Telegraph Company and the Marconi International Marine Communication Company, which appointments he held until his death. He was also a director of various other companies, including the Spanish and General Corporation, the Wireless Press, the Marconi International Code Company, the Relay Automatic Telephone Company, the British Niclausse Boiler Company, Eastwoods, the Never-Stop Transit Company, and the Still Engine Company.

He was elected a member of The Institution in 1895, became a Member of Council in 1915, and was elected Vice-President in 1925, but did not live to take office as such. He was Honorary Secretary of the Committee appointed in 1896 to consider the Standards of Efficiency of Heat-Engines, and wrote the introductory note[2] to the Committee's report. He succeeded Sir William Anderson as Chairman of the Committee, appointed by The Institution in 1897, on Steam-Engine and Boiler Trials, and was a member of the Committee on Internal-Combustion Engines. At the time of his death he was Chairman of the Heat-Engine Trials Committee, which since 1922 has had under consideration the revision and extension of the work of the previous Institution Committees on Heat-Engines.

In addition to the Paper already mentioned, he presented in 1896 a Paper on "The Thermal Efficiency of Steam-Engines,"[3] for which he was awarded a Telford Medal. In 1897 he contributed a Paper on "The Decimal System in Engineering Measurement."[4] He delivered a lecture to the London Students' Association in

[1] Minutes of Proceedings Inst. C.E., vol. clxii, p. 371.
[2] *Ibid.*, vol. cxxxiv, p. 278.
[3] *Ibid.*, vol. cxxv, p. 182.
[4] *Ibid.*, vol. cxxx, p. 218.

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1923 on "Recent Developments in Wireless Telephony," and in 1925 he gave the James Forrest Lecture on "Heavy-Oil Engines: Some Outstanding Questions relating to Large Engines of the Self-Ignition Type."

He was admitted to membership of the Institution of Mechanical Engineers in 1894, became a member of council in 1910, and was President in the Sessions 1920 and 1921. In addition to the several Papers he presented to that Society, he delivered the Thomas Hawksley Lecture in 1907 on the subject of Heat-Engines. He was Chairman of two standing committees and also of the special committee on Steam-Nozzle Research.

His earliest association with a scientific society was his admission to membership of the Society of Telegraph Engineers and Electricians (now The Institution of Electrical Engineers) in 1885, in which year he was awarded the first Fahie Premium of the Society for an account of his work in the Ordnance Department at Southampton. He was also a member of the Institution of Naval Architects, of the Iron and Steel Institute, of the Institution of Gas Engineers, of the Junior Institution of Engineers (of which he was President in the Session 1922–23), and of the American Society of Mechanical Engineers. He served on the General Board of the National Physical Laboratory, and was a member of several sub-committees of the British Engineering Standards Association, as well as of the Wireless Telegraphy and Gaseous Explosions Committees of the British Association.

On the outbreak of war in 1914 he volunteered his services and was appointed Honorary Engineering Adviser to the Director of Fortifications and Works. In this capacity he was engaged in a wide range of important duties, of some of which mention is made in his Presidential Address to the Institution of Mechanical Engineers. He went to Flanders in 1915 to devise a scheme of trench drainage, which was adopted with success, and again in 1918 he proceeded overseas to report, in conjunction with the late Sir Maurice Fitzmaurice, Past-President, on the possibility of inundating certain portions of the western front. He was made a Companion of the Bath in 1916, and in 1920 his services as member of the Honorary Valuation Committee were recognized by the conferment of the C.B.E.

Besides contributing to the Proceedings of numerous scientific societies, he found time for other literary work of a technical nature. He translated, from the German, Ritter's work on "Bridges and Roofs," wrote "The Energy Chart and its Practical Application to Reciprocating Steam Engines," and was the anonymous author of Part IV of Rivington's "Building Construction." In collaboration with Mr. W. J. Marshall he wrote a work on "Gas Engines."

He married in 1876 Elizabeth Mary, eldest daughter of General E. L. Pym, Royal Marines. Of their five children, two were sons. The younger son died in infancy, and the elder, Colonel C. E. P.

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Sankey, D.S.O., R.E., Assoc. Inst. C.E.—who at one time assisted him in his consulting practice—survives him.

Captain Sankey was peculiarly well endowed for a wide range of professional scientific work and he devoted himself with the utmost generosity to the advancement of his profession in every possible way. To the many who were so fortunate as to know him, his death was a great personal loss. His ability and tireless energy gained their respect as fully as his courtesy, charm of manner and humour won their affection. He passed away suddenly from heart failure at his residence at Ealing, on the 3rd October, 1925, in the midst of the activities that thronged his busy life.

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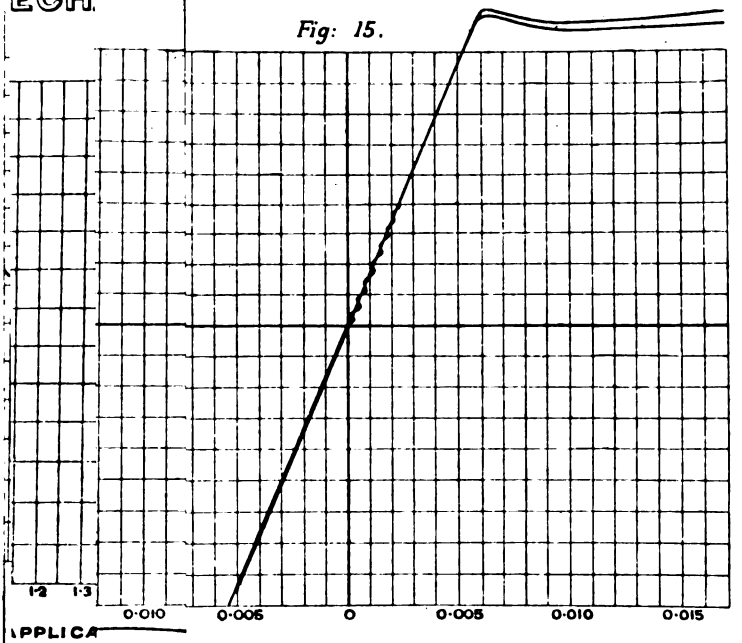
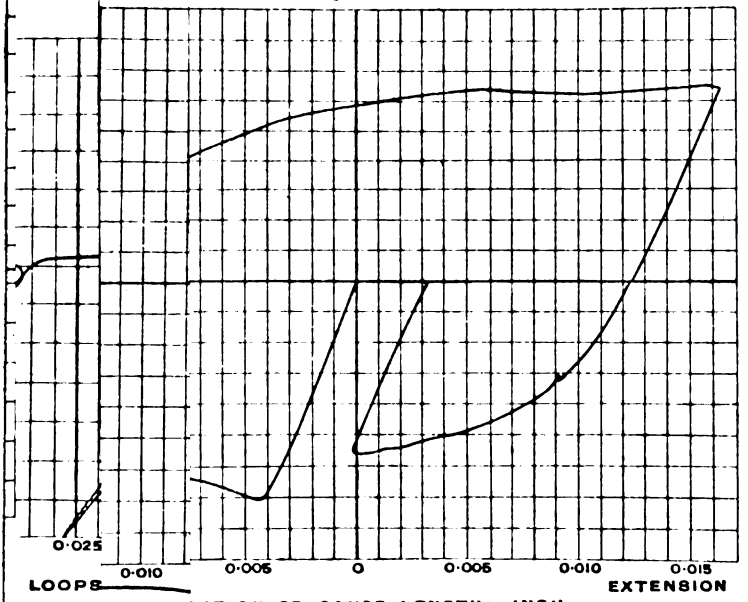
Fig: 15.



APPLICA

ALTERATION OF GAUGE-LENGTH: INCH
NICKEL VERSED AND SUPERPOSED ON FIG. 12

Fig: 16.



ALTERATION OF GAUGE-LENGTH: INCH
FIEL. ALTERNATING PUSH AND PULL.

Proceeding

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PLATE 5.
MONTE-VIDEO WATERWORKS.
BOOSTING-STATION.

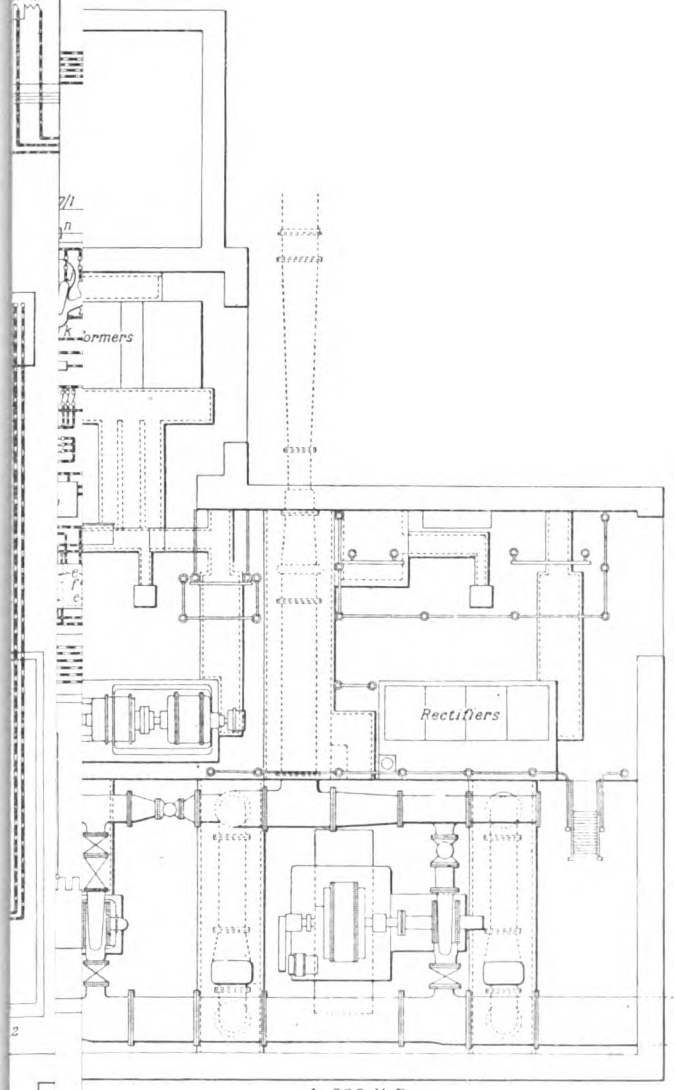

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PLATE 9.

TRANMERE DOCK ENTRANCE.

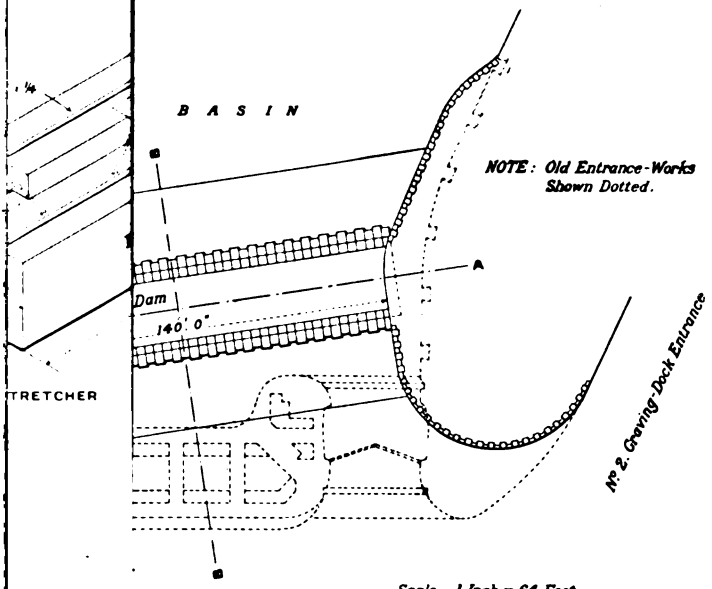
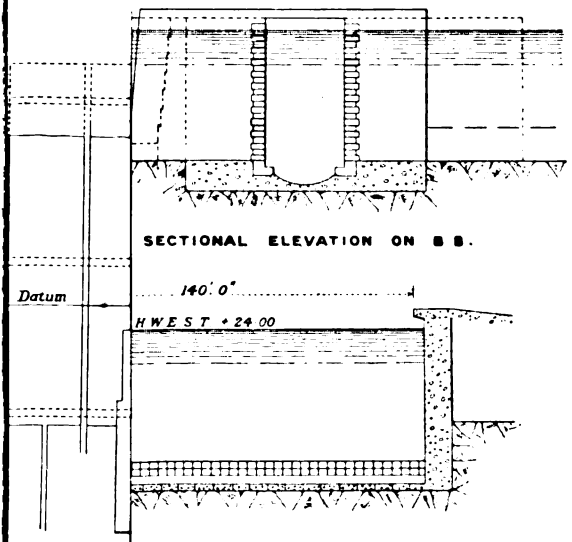


Fig. 6.

Scale: 1 Inch = 64 Feet.

Feet 20 10 0 20 40 60 80 100 Feet



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